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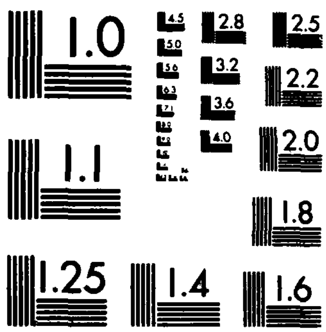

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CONNECTICUT COASTAL BASIN

DERBY-SHELTON, CONNECTICUT

AD-A142 592

LAKE HOUSATONIC DAM AND DIKE
CT 00026
CT 01714

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1981

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The Lake Housatonic Dam, formerly known as the Derby Dam, was completed in Oct., 1870 to facilitate river traffic on the Housatonic River and supply water to nearby factories. The entire facility consists of a 400 ft. long earthfill dike along the left bank; a gatehouse at the left abutment that regulates flow into an industrial water supply canal; a 675 ft. long by 23 ft. high spillway section spanning the river; and a second gatehouse and boat lock at the right abutment. The earthfill dike has a maximum height of approximately 10 ft. and is 15 ft. wide at the crest. The tota height of the dam from the downstream toe of the spillway section to the top of the earthfill dike is approximately 40 ft.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION. CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM. MASSACHUSETTS 02254

REPLY TO
ATTENTION OF:
NEDED

SEP [illegible]

Honorable William A. O'Neill
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor O'Neill:

Inclosed is a copy of the Lake Housatonic Dam & Dike (CT-00026 & CT-01714) Phase I Inspection Report, prepared under the National Program for Inspection of Non-Federal Dams. This report is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. I approve the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is vitally important.

Copies of this report have been forwarded to the Department of Environmental Protection, and to the owner, Connecticut Light & Power Co.. Copies will be available to the public in thirty days.

I wish to thank you and the Department of Environmental Protection for your cooperation in this program.

Sincerely,

C. E. Edgar III

C. E. EDGAR, III
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

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COPY
INSPECTED
2

A-1

LAKE HOUSATONIC DAM AND DIKE

CT 00026

CT 01714

CONNECTICUT COASTAL BASIN

DERBY-SHELTON, CONNECTICUT

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No.: CT 00026, CT 01714

Name of Dam: Lake Housatonic Dam and Dike

Town: Derby-Shelton

County and State: New Haven-Fairfield, Connecticut

Stream: Housatonic River

Date of Inspection: July 7 and 15, 1981

BRIEF ASSESSMENT

The Lake Housatonic Dam, formerly known as the Derby Dam, was completed in October, 1870 to facilitate river traffic on the Housatonic River and supply water to nearby factories. The entire facility consists of a 400-foot-long earthfill dike along the left bank (average crest El. 39.4 NGVD); a gatehouse at the left abutment that regulates flow into an industrial water supply canal; a 675-foot-long by 23-foot-high spillway section spanning the river; and a second gatehouse and boat lock at the right abutment. The earthfill dike has a maximum height of approximately 10 feet and is 15 feet wide at the crest. The total height of the dam from the downstream toe of the spillway section to the top of the earthfill dike is approximately 40 feet.

The left gatehouse contains three 8-foot by 8-foot gates that may be operated manually or with a portable device that is driven by an electric motor. Discharge from the gatehouse flows through a canal, paralleling the river, before it is returned to the Housatonic River approximately 2,230 feet downstream. The spillway section is curved in plan and has an average crest elevation of approximately 23.7 (top of flashboards El. 25.2). The downstream face of the spillway is concave and terminates at an apron. The gatehouse and boat lock, located at the right abutment, regulate flow into a canal which is approximately 80 feet wide and 3,200 feet long. An emergency spillway, located adjacent to the spillway

section and extending 145 feet downstream from the gatehouse, discharges excess flow from the canal into the river. Approximately 1,680 feet downstream of the gatehouse, in the left bank of the canal, is a lock system consisting of three gates which leads back to the Housatonic River. The canal is formed by an earthfill embankment on the left side and a vertical concrete wall, which retains a railroad embankment, along the right side.

The visual inspection of the Lake Housatonic Dam indicated that the structure is in fair condition. Deterioration of the right abutment, as evidenced by seepage through the masonry joints and the missing stone blocks near the spillway dam flashboards, may adversely effect the integrity of the abutment and the spillway section by permitting water to enter the interior of the structures. In addition, erosion; the growth of trees and brush; the lack of riprap slope protection; and the absence of protective ground cover on the dike adjacent to the left gatehouse will promote the deterioration of the embankment and ultimately diminish the stability of the structure if allowed to continue. The site also lacks the means to drawdown the impoundment.

The Lake Housatonic Dam has a top of the dam storage capacity of 10,900 acre-feet (ac-ft), is approximately 40 feet in height, and is considered to be INTERMEDIATE in size. The failure of the dam could cause the loss of more than a few lives; therefore, the dam has been classified as having a HIGH hazard potential. The test flood for the Lake Housatonic Dam is the Probable Maximum Flood (PMF). The peak inflow is 252 cubic feet per second per square mile (cfs/sg. mi.) or 396,000 cubic feet per second (cfs) and the peak outflow is 394,000 cfs. The capacity of the spillway dam, with the water surface at the top of the dam, is 155,500 cfs or 39 percent of the routed test flood outflow. During the test flood the dam will be overtopped by 7.5 feet.

It is recommended that the owner retain the services of a qualified registered professional engineer to investigate those areas where deterioration of masonry was found and institute corrective measures;

investigate the downstream toe of the spillway section; restore the dike along the left side of the impoundment; ascertain the need for and the means to provide drawdown of the pool; perform a detailed hydrologic - hydraulic investigation to assess the potential of overtopping the spillway, dike, and abutments; determine the effect of the silt layer covering the upstream face of the spillway section on the stability of the structure; and develop a program for the possible utilization of the existing canal systems as low level outlets.

The recommendations and remedial measures outlined above and discussed in Section 7 should be instituted within one (1) year of the owner's receipt of this report.

Reynold A. Hokenson

Reynold A. Hokenson, P.E.
Project Manager
International Engineering Company, Inc.

COMMONWEALTH OF MASSACHUSETTS
REYNOLD
A.
HOKENSON
No. [illegible]
REGISTERED
PROFESSIONAL ENGINEER

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm

event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

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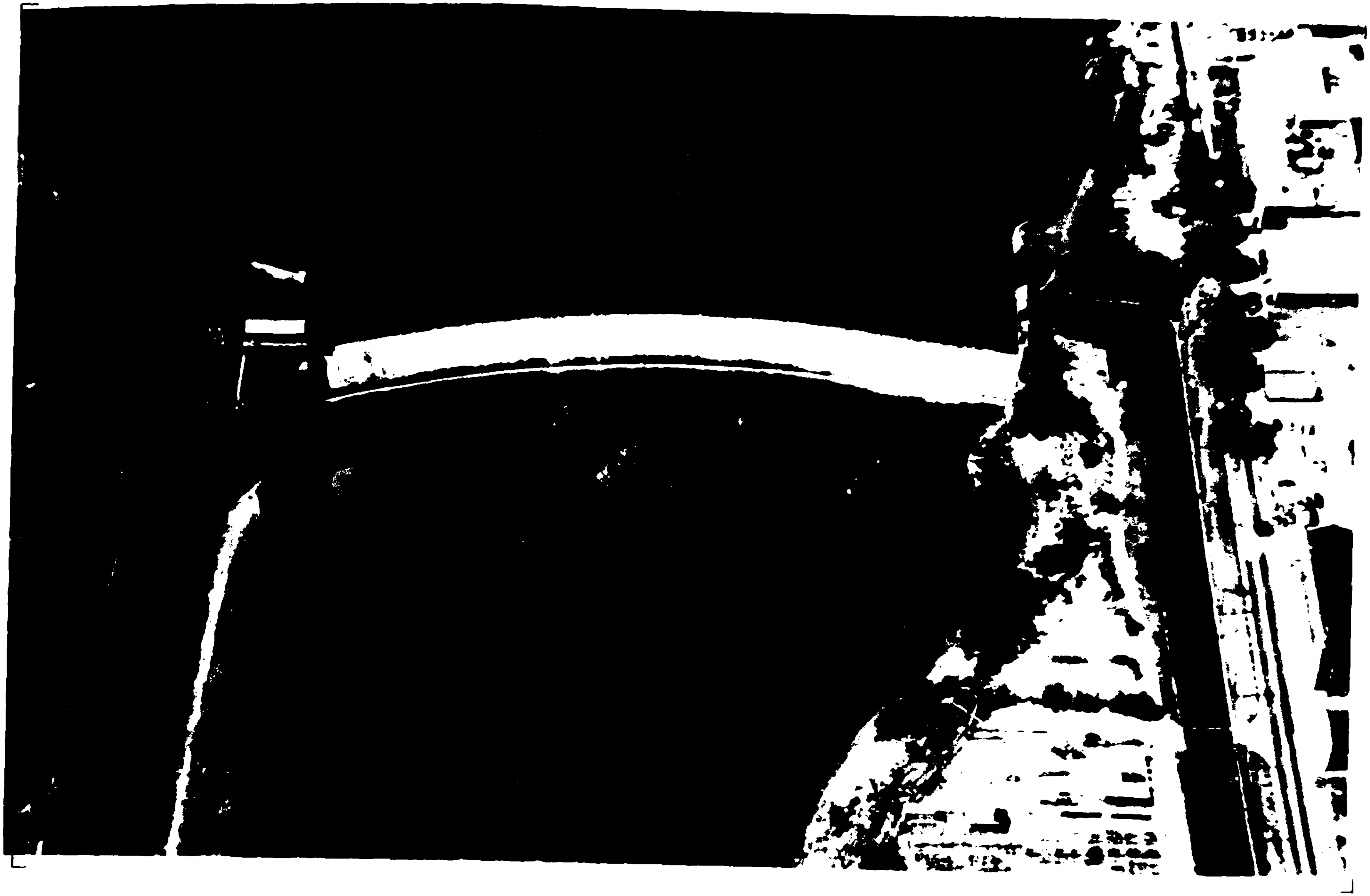
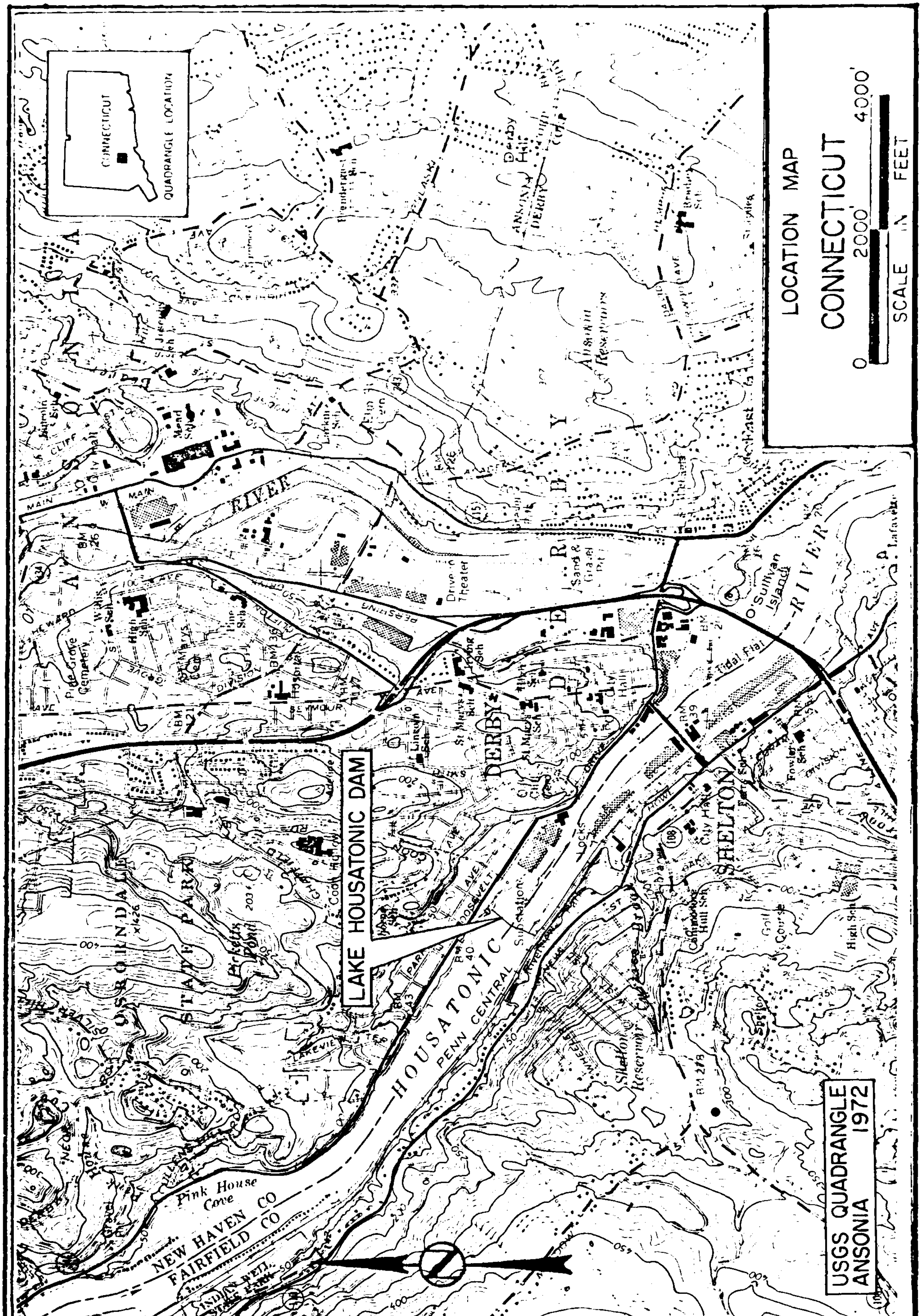


OVERVIEW PHOTO-LAKE HOUSATONIC DAM

AUGUST 1981


LOCATION MAP
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SCALE IN FEET
USGS QUADRANGLE
ANSONIA 1972
LAKE HOUSATONIC DAM
CONNECTICUT
QUADRANGLE LOCATION
HOUSATONIC
RIVER
DERBY
SHELTON
NEW HAVEN CO
FAIRFIELD CO
PENN CENTRAL
Pink House Cove
O'Sullivan Island
Tidal Flat

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

LAKE HOUSATONIC DAM AND DIKE

SECTION 1: PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England region. International Engineering Company, Inc., has been retained by the Corps' New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to International Engineering Company in a letter dated June 18, 1981, from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW33-81-C-0015 has been designated by the Corps for this work.

b. Purpose of Inspection Program - The purpose of the program are to:

(1) Perform technical inspections and evaluations of non-Federal dams to identify conditions requiring correction in a timely manner by non-Federal interests.

(2) Encourage and prepare the States to quickly initiate effective dam inspection programs for non-Federal dams.

(3) Update, verify, and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I Inspection Report includes:

(1) Gathering, reviewing, and presenting all available data as can be obtained from the owners, previous owners, the state, and other associated parties.

(2) A field inspection of the facility detailing the visual condition of the dam, embankments, and appurtenant structures.

(3) Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

(4) An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgment on the safety or stability of the dam other than on a visual basis. The purpose of the inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on the Housatonic River on the border of Shelton and Derby, Fairfield-New Haven Counties, Connecticut. The Lake Housatonic Dam is the last dam on the Housatonic River before the confluence of the Housatonic and Naugatuck Rivers. The location of the dam is defined by the coordinates latitude N 41°19.5' and longitude W73°6.2' on the Ansonia, Connecticut, USGS Quandrangle Map.

b. Description of the Dam and Appurtenances - The Lake Housatonic Dam consists of a 675-foot-long spillway section which is curved in plan; a 400-foot-long earthfill dike along the left bank of the impoundment extending upstream from the spillway section; and two canals located along both banks of the Housatonic River projecting downstream of the dam. Flow into the canals is regulated by the impoundment's outlet works located in the gatehouses at the abutments of the spillway section. The total height of the dam from the downstream toe of the spillway section to the top of the earthfill dike is approximately 40 feet.

The main spillway section (Average Crest El.23.7 NGVD) is a 675-foot-long and 23-foot-high cut stone and concrete structure that is arched in plan. (Note: All elevations are referenced to the National Geodetic Vertical Datum). The spillway crest is a 8-foot-wide slab with 1.5-foot-high flashboards. The upstream slope of the dam was measured to be approximately 1V:4H. The concrete downstream face of the spillway section is concave and terminates at an apron. Both spillway abutments are composed of vertical stone masonry walls, which also form part of the gatehouse substructure. The top of the left abutment is at El. 35.87 and the elevation of the right abutment is 33.91.

The earthfill dike along the left bank of the impoundment is approximately 400 feet long and has a maximum height of 10 feet. Measurements of the upstream and downstream slope yielded slopes of 1.5H:1V and 1.2H:1V, respectively. The crest width of the dike is approximately 15 feet (crest elevation varies from El. 38.9 to El. 39.8).

The masonry gatehouses located at the abutments of the main spillway regulate flow into the canals. The canal along the left bank of the Housatonic River serves primarily as a source of industrial water. Flow into the left canal is controlled by three 8-foot by 8-foot gates. The gate hoist mechanisms, contained in the gatehouse superstructure, may be operated manually or by an electrically driven portable gate operator. The canal is approximately 40 feet wide by 2,230 feet long and

is formed by vertical masonry side walls (Canal Invert at the gatehouse is El. 13.87). Flow in the canal passes through four 8-foot diameter by 300-foot-long corrugated metal conduits approximately 1,450 feet from the gatehouse where the canal intersects Roosevelt Drive. At the end of these conduits the canal is again an open channel which is parallel to Roosevelt Drive. An overflow weir, approximately 400 feet downstream from the 8-foot diameter conduits defines the end of the canal. Discharge from the canal can occur over the weir; through the gated outlet at the weir; or through one of the three industrial water intakes adjacent to Roosevelt Drive. Flow over the weir passes through a 6-foot-high by 9-foot-wide masonry culvert under Roosevelt Drive before the confluence of the canal and the Housatonic River is reached, 80 feet from the overflow weir.

The canal parallel and adjacent to the right bank of the Housatonic River originally served to divert boat traffic through a series of locks around the Lake Housatonic Dam. The canal is formed by a concrete wall which retains a railroad embankment on the right side and an earthfill dike on the left side. The earthfill dike is approximately 1,700 feet long; 20 to 25 feet high; and is 15 feet wide at the top. The upstream face of the dike is faced with stone and sloped at about 1V:1H. Flow into the canal is reguated by five gates located in the gatehouse adjacent to the spillway section (Canal Invert at the gatehouse is El. 10.71). These gates may be operated manually or with an electric motor which may be engaged through a series of belts and friction clutches with the gate hoist stems. The water level within the canal is maintained at a constant level by a 145-foot-long spillway (crest El. 21.79) that extends parallel to the river and is located downstream of the gatehouse. The canal spillway crest has been raised to El. 22.59 through the addition of an 0.8-foot-high flashboard. Flow over the canal spillway crest plunges vertically approximately 20 feet to the floor of the Housatonic River. The canal level may be lowered by opening the gated outlet located within the right abutment wall between the canal spillway and the spillway section (see Appendix B pg. B-37). The control mechanism for this outlet is located on the canal side of the right abutment. A boat lock between the right bank and the gatehouse provides access to the canal from Housatonic Lake. The lift from the canal to

lake level is approximately 2.6 feet measured from the top of the canal spillway flashboard to the top of the spillway section flashboards. The downstream entrance to the canal consists of two boat locks located in the left bank of the canal, 1,680 feet from the gatehouse. The canal continues downstream of the second set of locks as an open channel for approximately 350 feet to a bridge crossing. The bridge is constructed over two 72-inch diameter corrugated metal pipes (CMP) which convey flow downstream to the remainder of the canal. The canal continues 185 feet beyond the bridge as an open channel at which point it is routed through two 72-inch diameter by 565-foot-long CMPs. The remainder of the canal consists of a 420-foot-long open channel; two 72-inch diameter by 600-foot-long CMPs; a 48-inch diameter by 965-foot-long CMP; and a 90-foot-long by 24-inch diameter drain pipe. From the available plans of the area, fifteen industrial water supply intakes were identified between the downstream locks and the end of the 24-inch CMP.

c. Size Classification - The size classification is based on the height of the dam above the natural streambed or the maximum storage potential of the reservoir, which is defined by a pool at the level of the dam crest. The size classification of the dam is determined by the criteria that yields the larger size category. Lake Housatonic has a maximum potential storage capacity of 10,900 ac-ft and a height of 40 feet, which are within the established limits for the Intermediate size category, for storage (1,000 to 50,000 ac-ft) and height (40 to 100 feet).

d. Hazard Classification - HIGH - The hazard classification is based on the estimated loss of life and the anticipated property damage due to a dam breach when the water surface within the impoundment is at the top of the dam. The prefailure outflow over the spillway section is 155,500 cfs with 7,500 cfs of additional flow passing at the abutments. As a result, a substation and at least 6 factory complexes will be flooded to the depth of 2 feet within the initial impact area. The failure of the spillway section will cause the water surface elevation to rise 2.2 feet; thus inundating 3 additional commercial structures along the left river bank to a depth of 1 to 2 feet. The severity of the

prefailure flooding would result in the evacuation of the downstream area. Consequently, any additional flooding caused by the failure of the spillway section would increase the economic losses in the impact area and could potentially cause the loss of a few lives.

If failure of the earthfill dike on the left side of the impoundment occurs, with the water surface at the top of the dam, a 4 to 5-foot-high surge will cross Roosevelt Drive and impact with approximately 9 commercial buildings 150 to 200 feet from the dike, thus contributing to the hazard potential (see Appendix D, sheet D-30). Failure of this structure could potentially cause the loss of more than a few lives.

Since, no prefailure flooding is anticipated in the structures downstream of the dike, these structures would be occupied during flood periods. As a result, the failure of the dike could potentially cause the loss of more than a few lives. Therefore, the Lake Housatonic Dam has been classified as having a HIGH hazard potential.

e. <u>Ownership</u> -
The Connecticut Light and Power Company
Susbsidiary of Northeast Utilities Service Company
P.O. Box 270
Hartford, Connecticut 06101
(203) 666-6911

f. <u>Operator</u> -
Fossil/Hydro Production Department
Northeast Utilities Service Company
R.A. Reckert
(203) 666-6911

David Goddard
Hydroelectric Supervisor Housatonic River
(203) 355-1153
(203) 666-6911 Ext. 363

g. Purpose - Lake Housatonic is currently used for recreation and as an industrial water supply.

h. Design and Construction History - Funds for the construction of the dam currently known as the Lake Housatonic Dam (formerly The Derby Dam) were obtained in 1866. The dam was designed by Wm. E. Worthen of New York and constructed under the supervision of Henry J. Potter (Superintendent of Construction). The masonry and timber crib structure was completed in October, 1870. In the spring of 1891, a 210-foot-wide breach formed at the eastern side of the dam. The dam was repaired under the direction of Engineer D.S. Brimsmade during the summer of 1891. The reparations included: lengthening the dam, reconstructing the breached area with a dam of a "substantially wider" cross-section, and increasing the width of the apron over the remainder of the dam. Repairs were again made on the dam in 1948 after extensive wear and erosion were found on the wooden apron of the dam. That same year portions of the apron were renovated and the wooden planking replaced with concrete under the direction of D.M. MacWilliam, Hydraulic Engineer, The Connecticut Light and Power Company. The remainder of the wooden apron planking was replaced in a similar manner in 1952 by C.W. Blakeslee and Sons. See "A Short History of The Derby Dam", by Raul de Brigard, 7/81; and "Derby Dam" dated 12/24/80 in Appendix B for more detailed historical accounts.

i. Normal Operational Procedures - The water level in Lake Housatonic is maintained at the top of the flashboards (El. 25.2) for recreational purposes in the summer months. Flow through the canals is continuous, according to a representative of the owner, so as to provide an industrial water supply for the remaining users.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area tributory to Lake Housatonic Dam is 1,574 square miles. Headwaters lie in the Taconic Range reaching as far north in latitude as Albany, New York. The source rises to an elevation (maximum) of slightly over 3,000 feet on Brodie

Mountain, Massachusetts. The drainage basin is approximately 91 miles from the source to the Lake Housatonic Dam, thereby indicating part of Massachusetts (Pittsfield), portions of New York, and most of Western Connecticut, except the coastal draining area of the Southwestern portions of the state.

b. Discharge at Dam Site - Discharge normally occurs over the main spillway section and through the gate openings in the gatehouses.

(1) The outlet works from the site consist of the gate openings and associated canals at the left and right abutments of the main spillway. An estimation of the discharge capacity of these waterways was not performed due to a lack of data concerning the downstream canal outlets, and their unreliability to perform as effective outlets.

(2) The maximum known discharge (75,800 cfs) was recorded on October 16, 1955 at USGS gage No. 01205500 at Stevenson, Connecticut. According to available records for the dam site this flood caused the lake water surface to rise to El. 32.0 NGVD and a tailwater elevation of 23.2 NGVD (See Appendix B; pg. B-11).

(3) Ungated capacity of the spillway is 155,500 cfs at elevation 40.0.

(4) Ungated spillway capacity at test flood elevation (47.5) is 274,400 cfs.

(5) Gated spillway capacity at normal pool elevation - N/A.

(6) Gated spillway capacity at test flood elevation - N/A.

(7) Total spillway capacity at test flood (elevation 47.5) is 274,400 cfs.

(8) Total project discharge at top of dam (elevation 40.0) is 163,000 cfs.

(9) Total project discharge at test flood (elevation 47.5) is 394,000 cfs.

c. <u>Elevations</u> (feet above NGVD)

(1)	Streambed at toe of dam (approximate)	0.0
(2)	Botton of cutoff	Unknown
(3)	Maximum tailwater	24.9 August 19, 1955
(4)	Normal pool	25.2
(5)	Flood-control pool	N/A
(6)	Spillway crest	23.7
	Top of Flashboards (temporary)	25.2
(7)	Design surcharge (original design)	Unknown
(8)	Top of dam	40
(9)	Test flood surcharge	47.5

d. <u>Reservoir</u> (length)

(1)	Normal pool	4 miles
(2)	Flood-control pool	N/A

(3)	Spillway crest pool	4 miles
	Top of Flashboards (temporary)	4.1 miles
(4)	Top of dam	5 miles
(5)	Test flood pool	6 miles

e. Storage (acre-feet)

(1)	Normal pool	4,000
(2)	Flood-control pool	N/A
(3)	Spillway crest pool	4,000
	Top of Flashboards (temporary)	4,020
(4)	Top of dam	10,900
(5)	Test flood pool	15,000

f. Reservoir Surface (acres)

(1)	Normal pool	350
(2)	Flood-control pool	N/A
(3)	Spillway crest	350
	Top of Flashboards (temporary)	360
(4)	Top of dam	470
(5)	Test flood pool	630

g. Dike

(1)	Type	Earthfill dike
(2)	Length	400 ft.
(3)	Height	10 ft.
(4)	Top Width	15 ft.
(5)	Side Slopes	1.5H:1V upstream and 1.2H:1V downstream
(6)	Zoning	Unknown
(7)	Impervious Core	Unknown
(8)	Cutoff	Unknown
(9)	Grout Curtain	Unknown
(10)	Other	None

h. Diversion and Regulating Tunnel N/A

i. Spillway

(1)	Type	Broad-crested weir
(2)	Length of Weir	675 ft.
(3)	Crest elevation	23.7
	Top of Flashboards (temporary)	25.2
(4)	Gates	None

(5)	U/S Channel	Lake Housatonic
(6)	D/S Channel	Housatonic River

j. <u>Regulating Outlets - Gatehouses</u>

(1)	Invert Elevation (at gatehouse)	Right: 10.71; Left: 13.87
(2)	Size (Gate Openings)	Right: Unknown, Left: 3 @ 8 feet by 8 feet
(3)	Description	Sluice gates
(4)	Control Mechanism	Hand or electrically operated
(5)	Other	Boat locks in right canal and low level canal outlet at right abutment (Note: invert elevations and condition of these outlets are unknown).

SECTION 2: ENGINEERING DATA

2.2 DESIGN DATA

No design data were available for the Lake Housatonic Dam.

2.2 CONSTRUCTION DATA

A historical account describing the period of construction and the quantities of material used in the construction of the dam were available. Historical records indicate that major discrepencies were found in the location of ledge indicated during preliminary exploration and what was discovered to actually exist during the excavation of the foundation. Consequently, a gravel foundation was placed for the masonry dam. Construction was interrupted by cofferdam failures in 1867, 1868, and 1869. During the 1869 mishap, a 160-foot portion of the dam was swept away. See Appendix B for full history of dam.

2.3 OPERATION DATA

The lock system in the right canal is no longer used and the gates are in the closed position. The intake gates in the right and left gatehouses are currently used to maintain water in the canals since these canals are still used for industrial water supply. There are no regularly scheduled operations performed at the dam.

2.4 EVALUATION OF DATA

a. Availability - The bulk of the existing information concerning the Lake Housatonic Dam was made available by Northeast Utilities Service Company. This information included historical data, site topography, a stability analysis, tailwater curve, photographs, past operation records, cross-sections of the spillway section, and a Preliminary Permit Application to the Federal Energy Regulatory Commission for the hydroelectric development of the site. The State of Connecticut Water

Resources Department provided a data inventory sheet. Access to the site for the inspection was granted by the owner, and a representative was provided for consultation.

b. Adequacy - The available data supplemented by the inspection performed by International Engineering Company engineers was more than sufficient to complete the Phase I Inspection of Lake Housatonic Dam.

c. Validity - The field inspection indicated that the visible external features of the Lake Housatonic Dam are similar to those described in the available information. A check of the available tailwater curve, provided by Northeast Utilities Service Company in the Protrans Reconnaissance Study, was performed by International Engineering Company engineers using the Standard Step Method. The tailwater elevation at the Lake Housatonic Dam was calculated to be approximately 5 feet greater than what was stated in the available study (see Appendix D; pgs D-17, D-18, D-19 and D-20).

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General — Field inspections of the Lake Housatonic Dam were conducted on July 7 and 15, 1981, and areas requiring repair and maintenance were identified. As a result, the general condition of the facility has been determined to be fair. During the first inspection, the reservoir level varied from 0.35 to 1.35 feet above the flashboards (El. 26.22 to El. 27.57); consequently, the downstream apron was not inspected until the second visit at which time the upstream hydroelectric power station was shut down and there was no flow over the spillway section. Flow was observed through both gatehouses and over the right canal spillway during both inspections.

b. Dam — The downstream face of the spillway section is covered with a concrete slab (Photos 13 and 14). Several construction joints were visible, but only one seemed wider and, therefore, more pronounced. This joint is located approximately 30 feet from the left abutment and is visible in the foreground of Photo 14. The crest slab of the spillway section appears intact with the exception of some minor spalling (Photo 13). Variations in the crest elevation of 0.32 feet were indicated on the photogrammetric survey map (2-foot contours, scale 1:50) provided by the owner. The only observable seepage on the downstream face originates from the flashboards. However, an inspection report, dated July 11, 1975, listed two small jets of water emanating from the crest slab (see Appendix B, pg B-32).

During both inspections, the downstream toe of the spillway apron was inaccessible; therefore, this portion of the dam was not inspected. According to historical accounts and conversations with a representative from the owner, the river bottom consists of gravel. Boulders were noted, however, on the left side approximately 50 feet beyond the apron. The last inspection of this area was performed in November, 1979 and no scouring of the toe was found (see Appendix B pg B-29).

The only observable deterioration of the spillway section was found at the right masonry abutment where some stone blocks were missing and flow was being diverted between the abutment and the end of the flashboards. The left abutment appeared to be sound and no deterioration of the masonry was observed. The upstream slope of the spillway dam is reportedly covered by a 6 to 8-foot thick layer of silt and clay which forms a natural seepage barrier. Two piezometers were driven through the silt and clay layer and into the upstream slope of the spillway section to measure uplift pressures at the base of the spillway and determine the effectiveness of the seepage barrier. The available piezometer readings indicate that the uplift pressures on the upstream portion of the spillway are considerably less than the full hydrostatic pressure created by the current pond elevation (see Appendix B; pg B-4).

c. Appurtenant Structures — The right gatehouse (Photos 8 and 9) appeared to be sound and no deterioration of the masonry in the superstructure was observed. Access to the interior of the gatehouse is limited and can be gained only through the door on the east side of the structure. This door is normally locked and the key is held by the owner. The gate hoist mechanisms have been greased recently and are reportedly operable. The electrically driven gate hoist system is reported to be operable (Photo 11). The gates were completely submerged and, as a result, an evaluation of their condition was impossible. At the time of the inspection, there was flow through the gate openings. The mortar joints in the gatehouse foundation showed signs of deterioration, and calcite deposits were evident on the downstream face of the foundation. A seepage "jet" emanating between the masonry foundation blocks was observed adjacent to the spillway section. Discharge through this joint was estimated to be 20 to 25 gallons per minute (gpm); clarity of the seepage flow was indeterminable.

The boat lock gates between the right gatehouse and the railroad embankment (Photos 9 and 10) were closed and no significant deterioration or leakage was observed. In addition, the masonry side walls bordering the lock were intact.

The canal spillway was discharging during both site visits. The 0.8-foot-high flashboard on the spillway crest was firmly anchored and showed no signs of decay. The spillway appeared to be constructed of stone masonry, however, the crest section is concrete. A small foot bridge founded on the spillway crest provided access from the canal dike to the gatehouse. At the canal spillway right abutment, several masonry blocks were missing near the crest. In addition, seepage on the downstream side of this abutment was noted flowing down the masonry to the stone facing on the canal dike (Photo 12). The seepage flow was estimated to be 15 to 20 gpm; no particles were found in the clear discharge. The low level canal outlet gate operators did not appear to be operational. During the inspections, the outlet gate was closed and there was no flow through the opening.

The right canal (Photo 10) appeared to be silted and vegetation was noted growing on the bottom. According to available records, the canal was last used for boat traffic in 1972. The concrete retaining wall, bordering the canal on the right side, was intact, and only scattered areas of efforescence were noted. Several trees ranging from 6 to 12 inches in diameter, growing along the top of the wall, were noted overhanging the canal. The earthfill dike, along the left bank of the canal, is overgrown with trees, ranging from 2 to more than 20 inches in diameter, and a dense layer of brush. The stone facing, on the river side, has been displaced by root systems and is missing in several areas. However, no sloughing or erosion was observed on the dike. The paved road surface on top of the dike is uneven and cracked. Spot elevations from the photogrammetric survey of the site along the crest of the dike revealed a variation in elevation of 1.2 feet along the length of the structure. The canal side of the dike appears stable, but several small trees (2 to 5 inches diameter) and a dense layer of brush were found growing on the slope. No riprap slope protection was found on the dike's canal slope.

The left gatehouse (Photo 1) superstructure and substructure appeared to be sound, and no signs of cracking or differential settlement were noted. However, efflorescence was noted on the foundation and the

spillway section abutment walls. The gatehouse entrance, located on the southern side of the structure, is normally locked and the key is held by the owner. The gate hoist mechanisms, within the left gatehouse, are reportedly operable and have been greased and maintained recently; one of the gate stems has also been replaced (Photo 2). The condition of the portable, electrically operated gate hoist is unknown. During the inspection, flow through the gate openings into the canal was observed. The condition of the gates was impossible to determine given the flow through the gate openings and the water level in the gate chamber. The left gatehouse is also used to store replacement flashboards and pins. The flashboards remain in place during the summer to maintain a recreational pool.

The left canal was flowing freely during both site visits. A 20-foot-wide breach in the left masonry canal wall, approximately 100 feet from the gatehouse where the canal turns south, was noted. Several large trees growing along the right bank of the canal were found in the reach before the Roosevelt Drive culverts (Photo 5). The four 8-foot diameter culverts under Roosevelt Drive were apparently free of obstructions. Silt has accumulated to a depth of approximately 1-foot in the culvert adjacent to the right canal bank. The remainder of the canal is bordered by a densely wooded slope on the left side and Roosevelt Drive on the right side. The canal terminates at an overflow weir which discharges into a masonry road culvert under Roosevelt Drive. The four gated outlets at the end of the canal have been abandoned and no longer appear to be operable (Photo 6). The short discharge channel leading to the culvert contains several trees growing on a small island. In addition, the earth slopes bordering the discharge channel are overgrown with trees and brush. At the time of the inspection, flow in the road culvert was unobstructed (Photo 7).

The earthfill dike extending upstream of the left gatehouse has been overgrown with vegetation (Photo 3) and shows signs of localized erosion and sloughing. Trees ranging from 2 to 24 inches in diameter were growing on the upstream and downstream slopes, and localized erosion at exposed tree roots was observed on the upstream slope. The slopes were

also overgrown with a dense layer of brush. At the left abutment of the dike an area approximately 10 feet by 15 feet has been eroded 4 to 5 feet into the embankment material. The ground cover in this area and especially in the vicinity of the transmission tower, which is founded on the dike, is sparce, and in some places nonexistent. Footpaths have been worn into the downstream slope and along the top of the dike. The paths on the downstream slope have been deepened by rain runoff from the top of the dike. The vertical concrete wall, located, on the upstream slope and approximately 70 feet from the gatehouse, was inspected. This structure is about 3 feet wide at the top by 30 feet long and shows signs of deterioration. The concrete surface is cracking and spalling and evidence of efflorescence was observed. In addition, a 2-inch wide crack extending the height of the wall was found at the center of the structure. Erosion of the dike's upstream slope was evident between this concrete wall and the gatehouse. The eroded area extends 40 feet downstream from the concrete wall and is entirely overgrown by small trees (2 to 6-inches diameter). The depth of erosion into the slope was impossible to determine due to the dense layer of vegetation. However, this area does appear on the topography provided by the owner (see Appendix C Photo location Plan). According to the available topography and field measurments, the crest elevation of the dike varies as much as one foot.

d. <u>Reservoir Area</u> — The area immediately surrounding the impoundment is moderately developed and is primarily residential and recreational. The banks of the reservoir appear to be stable despite the variation in water surface elevation due to the operation of the upstream hydroelectric power plant.

e. <u>Downstream Channel</u> — The downstream channel follows the natural path of the Housatonic River and originates at the toe of the spillway section. An island located within the channel and approximately 450 feet downstream from the dam, divides the river for about 1,000 feet (Photo 15). The confluence of the Housatonic and Naugatuck Rivers is located about 1.5 miles downstream of the Lake Housatonic Dam. The banks of the channel are bordered by industrial areas.

3.2 EVALUATION

Based on the visual inspection of the Lake Housatonic Dam, it has been determined that the facility is in fair condition. The following may influence the future condition and/or stability of the structures comprising the Lake Housatonic facility. The magnitude of repairs and efforts to restore the canals and associated structures (ie side walls; canal dike; intake and outlet works; etc.) will be contingent upon the investigation of a qualified registered engineer as to the potential of the gatehouses and canals to provide pool drawdown capabilities.

(1) Scouring of the gravel river bottom adjacent to the downstream toe of the spillway apron could undermine the apron and induce settlement and cracking of the structure.

(2) Flow between the spillway section flashboards and right masonry abutment may deteriorate the abutment and spillway by permitting water to enter the interior of the structures.

(3) The seepage "jet" on the right masonry spillway section abutment may originate from the gate chamber. This seepage may indicate the deterioration of the gatehouse substructure adjacent to the spillway.

(4) Seepage through the right canal spillway abutment and missing stone blocks may indicate the deterioration of the masonry joints. Extensive deterioration could adversely effect the stability of the canal spillway and the adjacent main spillway section abutment.

(5) The trees growing on the earthfill dike along the left side of the impoundment may be uprooted and severely damage the embankment. In addition, the growth of trees and brush may deteriorate the structure and induce seepage along the root networks.

(6) The lack of riprap on the dike's upstream slope and the absence of proper protective ground cover over the rest of the dike will result in further erosion of the embankment.

(7) Variations in the crest elevation of the dike may indicate undesirable foundation and embankment material settlements or surface erosion. This has reduced the freeboard and makes the dike susceptable to overtopping during periods of high project discharge.

(8) The absence of an operable canal drain or low level outlet in both canals will hinder repairs at the gatehouse and along the canal side walls.

(9) The breached section of the left canal training wall could progress under higher flow conditions and effect the capacity of this outlet from the lake.

(10) The trees growing on the earthfill dike bordering the right canal may be uprooted and severly damage the embankment. In addition, the growth of trees and brush may deteriorate the structure by allowing seepage to occur along the root networks.

(11) The trees overhanging the right and left canals will add to the accumulation of debris within the canals.

(12) The current condition of the downstream canal outlets will obstruct discharge from the canals.

SECTION 4: OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 OPERATIONAL PROCEDURES

a. General — Lake Housatonic is currently used for recreation and as an industrial water supply. The 1.5-foot-high flashboards on the spillway crest are used to add to the recreational pool. The flashboards remain in place during the summer and are restored as needed in spring. Discharge from the site normally occurs over the spillway section and through the canals.

b. Description of any Warning System in Effect — There is no formal downstream system currently in effect at the site.

4.2 MAINTENANCE PROCEDURES

a. General — Currently, no regularly scheduled maintenance procedures are performed at the site.

b. Operational Facilities — The gate hoist mechanisms in the left and right gatehouses have been lubricated and maintained recently. In addition, a gate stem in the left gatehouse has been replaced. The boat lock system in the right canal according to available information has been abandoned. Despite the current use of the outlet works in the gatehouses, there is no regular maintenance of these mechanisms.

c. Other — The owner is in the process of filing a Licensing Application before the Federal Energy Regulatory Commission with the intention of developing the hydroelectric potential of the site. As a result, significant modifications and alterations of the site are anticipated in the near future.

4.3 EVALUATION

The maintenance procedures currently employed at the site are

inadequate. Records documenting the operation and maintenance of the facility and providing a detailed account of the work and/or operations performed should be kept for future reference. In addition, a formal downstream warning system, emergency operations guidelines, and a program of annual technical inspection performed by a qualified registered professional engineer should be established. Remedial measures and maintenance recommendations are presented in Section 7.

SECTION 5: EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

5.1 GENERAL

The drainage area tributary to Lake Housatonic Dam is 1,574 square miles. Headwaters lie in the Taconic Range reaching as far north in latitude as Albany, New York. The source rises to an elevation (maximum) of slightly over 3,000 feet on Brodie Mountain, Massachusetts. The drainage basin is approximately 91 miles long, thereby the basin has an average width of only slightly more than 17 miles. The basin includes part of Massachusetts (Pittsfield), portions of New York, and most of Western Connecticut, except the coastal drainage area of the Southwestern portions of the state. The tailwater of Lake Housatonic Dam is tidal. The dam is approximately 11 miles from Long Island Sound and 1.3 miles upstream of the Housatonic-Naugatuck River confluence. Flow is regulated by Northeast Utilities at the Stevenson Hydropower Plant, 5.5 miles upstream of Lake Housatonic Dam. The Stevenson Dam impounds Lake Zoar.

The facility at Lake Housatonic consists of a spillway section across the Housatonic River; an earthfill dike extending upstream of the spillway section along the left bank of the impoundment; and two gatehouses located at the spillway section abutments. The gatehouses regulate flow into the canals which parallel the river downstream of the site. The structures comprising the Lake Housatonic Dam are in fair condition. The earthfill dike is overgrown with mature trees and lacks adequate slope protection on the upstream face. The masonry spillway section abutments and the canal spillway show signs of deterioration. In addition, obstructions and potential problem areas in the left and right canals were noted.

5.2 DESIGN DATA

No hydraulic or hydrologic design data could be found for the original dam.

5.3 EXPERIENCE DATA

The original dam, constructed in 1870, was partially breached in the spring of 1891. Restoration of the structure commenced during the summer of 1891. The maximum known discharge (75,800 cfs) was recorded on October 16, 1955, at the United States Geological Survey (USGS) Gage No. 0120550 at Stevenson, Connecticut. The headwater and tailwater elevations at the Lake Housatonic Dam were 32.0 and 23.2 respectively.

5.4 TEST FLOOD ANALYSIS

The maximum potential storage capacity (10,900 ac-ft) and the height (40 feet) of the Lake Housatonic Dam are within the limits established by the Corps in the "Recommended Guidelines for Safety Inspection of Dams", dated September, 1979, for the INTERMEDIATE size category. The hazard classification for the dam is HIGH, since there is the potential for the loss of more than a few lives due to the breach of the dam. Based on the size and hazard classifications, the recommended test flood for this dam is the Probable Maximum Flood (PMF). Due to the reservoir surcharge storage during the test flood, the PMF inflow will not be significantly attenuated. The PMF was obtained by doubling the Standard Project Flood (SPF) of 198,000 cfs. The peak inflow to the lake due to this flood in a 1,574 sq. mi. watershed is approximately 252 cfs/sq. mi. The test flood inflow (396,000 cfs) and resulting outflow (394,00 cfs) will cause the water surface elevation within the impoundment to rise to El. 47.5 or 7.5 feet above the top of the dam. The capacity of the spillway is 155,000 cfs with the water surface at the top of the dam (El. 40.0) or 39 percent of the routed test flood outflow.

5.5 DAM FAILURE ANALYSIS

Utilizing a conservation of momentum analysis because high tailwater conditions were suspected of limiting the maximum failure discharge and comparing the results with the "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", dated April, 1978, without tailwater

effects, as a check, the maximum failure discharge was determined (see Appendix D; pg. D-21). The prefailure outflow over the spillway section when the water surface is at the top of the dam is 155,500 cfs assuming the flashboards are not in place. An additional 7,500 cfs passes over the abutments and portions of the dike near the left gatehouse; thus making the total outflow from the site, prior to the breach, 163,000 cfs. The subsequent breach of the spillway section, encompasses 250 feet of the structure and results in a total outflow of 209,000 cfs.

The failure of the spillway section at the Lake Housatonic Dam will cause the water surface within the impact area to rise from El 25.8 feet at a total prefailure discharge of 163,000 cfs to El 27.95 after the failure. The prefailure stage within the initial impact area would inundate 6 factory complexes to a depth of approximately 2 feet. Following the dam failure, 3 additional commercial buildings would be flooded to a depth of 1 to 2 feet. Prefailure flooding due to discharge over the spillway section would result in the evacuation of those areas downstream of the dam. As a result, any additional flooding due to the failure of the spillway section would increae the economic losses in the downstream area and could potentially cause the loss of a few lives.

Assuming the top of the dam is at elevation 40, flow will also occur over a portion of the dike upstream of the left abutment and gatehouse. If failure occurs, a surge will cross Roosevelt Drive approximately 4 to 5 feet in height above street level. The surge will travel at a rate of 10-15 feet/second and impact with approximately 9 commercial buildings (first floor elevations approximately 1-foot above street level) 150 to 200 feet from the dike. No prefailure flooding is anticipated in these structures. As a result, the failure of the dike could potentially cause the loss of more than a few lives (see Appendix D; D-31). Therefore, the Lake Housatonic Dam has been classified as having a HIGH hazard classification.

SECTION 6: EVALUATION OF STRUCTURAL STABILITY

6.1 VISUAL OBSERVATION

The visual inspection of the dam did not reveal any indications of immediate stability problems. Deterioration of the right abutment, as evidenced by seepage through the masonry joints and the missing stone blocks near the spillway section flashboards, may adversely effect the integrity of the abutment and the spillway section by permitting water to enter the interior of the structures. In addition, erosion; the growth of trees and brush; the lack of riprap slope protection; and the absence of protective ground cover on the dike adjacent to the left gatehouse will promote the deterioration of the embankment. This deterioration will ultimately adversely effect the stability of the structure if allowed to continue.

6.2 DESIGN AND CONSTRUCTION DATA

Design calculations and detailed construction data were not available to assess the structural stability of all of the important water retaining structures. Historical accounts included in Appendix B describe the period of construction. A preliminary stability analysis of the spillway section, performed as a part of a hydroelectric reconnaissance study, is included in Appendix B. The criteria used in this study are not those commonly used by the U.S. Army Corps of Engineers.

6.3 POST-CONSTRUCTION CHANGES

The dam was breached in the spring of 1891 and reconstruction commenced during the summer under the direction of Engineer D.S. Brimsmade. Renovation of the spillway apron, which consisted of replacing the timber planking with a 9-inch layer of concrete was performed, on part of the dam, under the direction of D.M. MacWilliam, Hydraulic Engineer, of the Connecticut Light and Power Company in 1948. The remainder of the planking was replaced with concrete by C.W.

Blakeslee & Sons in 1952. Future modifications planned include the installation of a hydroelectric powerplant at the site.

6.4 SEISMIC STABILITY

The dam is in Seismic Zone 1, and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Condition – Based on the visual inspection of the site and past performance, the Lake Housatonic Dam is in fair condition. No evidence of structural instability was observed in the spillway dam, dike, gatehouses, or abutments. However, deterioration of masonry and seepage were noted at the right abutment of the spillway section and at the canal spillway abutment. In addition, several areas requiring maintenance and further investigation were identified.

Based on the "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", dated April 1978, and the hydraulic/hydrologic computations, the peak inflow and outflow for the test flood are 396,000 cfs and 394,000 cfs, respectively. The spillway capacity with the water surface at the top of the dam (El. 40.0 NGVD) is 155,500 cfs or 39 percent of the routed test flood outflow.

b. Adequacy of Information – The information available is such that an assessment of the condition and stability of the dam must be based largely on the visual inspection, past performance, a reconnaissance-type stability analysis, and sound engineering judgement. The owner of Lake Housatonic Dam currently has plans to develop the hydroelectric potential of the site and intends to submit a FERC license application in the Fall of 1981. Consequently, exploration of the site and a detailed evaluation of the condition and stability of the dam are currently in progress.

c. Urgency – It is recommended that measures presented in Sections 7.2 and 7.3 be implemented within one (1) year of th wner's receipt of this report.

7.2 RECOMMENDATIONS

It is recommended that the following items be undertaken by a

registered professional engineer qualified in dam design and inspection:

(1) Investigate the condition of the canal spillway and develop a program which should include, but not be limited to repairing the masonry and eliminating the seepage through the right abutment.

(2) Investigate the downstream toe of the spillway section to determine if scour and possibly undermining of the structure have occurred since the last inspection in 1979.

(3) Determine the source of the seepage jet at the right abutment of the spillway section and develop a program to correct the seepage.

(4) Repair the damaged portions of the spillway dam at the right abutment interface.

(5) Develop a program for the restoration of the dike bordering the left side of the lake. The trees, stumps, and roots on the dike should be removed and the resulting voids backfilled with a suitable compacted material. The crest of the dike should be leveled and reshaped by backfilling low points with a suitable compacted material. Grass should be planted over those areas of the embankment requiring backfilling to prevent future erosion. In addition, the eroded areas on the upstream face of the dike should be backfilled with a suitable compacted material. Those areas subject to wave action should be protected with riprap.

(6) Ascertain the need for and the means to provide drawdown of the pool. This investigation should assess the potential of the existing gates and canals for providing such an outlet.

(7) Perform a detailed hydrologic-hydraulic investigation to assess further the potential of overtopping the spillway, the dike, and the abutments, and the need for and the means to increase project discharge capacity. A verification of the tailwater rating curve should be a part of the hydrologic - hydraulic investigation.

(8) Develop an operations and maintenance manual for the facility.

(9) Investigate the silt layer on the upstream slope of the spillway dam and its effect on the stability of the structure.

(10) Perform an investigation to determine the operability of the canal outlets. Specifically, investigate the lower boat lock and low level outlet in the right canal and the gated outlet at the overflow weir in the left canal. Repair of the outlets should be considered so as to facilitate repairs at the gatehouses and the canal side walls.

(11) Repair the breached section of the left canal wall.

(12) Develop a program for the restoration of the earthfill dike bordering the right canal. The trees, stumps, and root systems should be removed and the resulting voids backfilled with a suitable compacted material. The displaced stone facing on the river side should be replaced and riprap provided on those areas susceptable to erosion on the canal side.

(13) A downstream outlet from the right canal should be provided if this waterway is to be considered for future use as a low level outlet from the lake.

The owner should implement the recommendations of the Engineer.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures — The following measures should be undertaken within one (1) year of the owner's receipt of this report and continued on a regular basis.

(1) A formal program of operation and maintenance procedures should be instituted and documented to provide accurate records for future reference.

(2) An "Emergency Action Plan" should be developed that will include: monitoring the site during flood periods; an effective preplanned downstream warning system; locations of emergency equipment, materials, and manpower; and authorities to contact.

(3) Institute a program of annual technical inspection by a qualified registered engineer.

(4) Insure unobstructed flow in both canals by removing overhanging trees and debris within the channel boundaries (ie excavate silt and clear trees growing in left canal discharge channel adjacent to Roosevelt Drive).

7.4 ALTERNATIVES

This study has identified no practical alternatives to the above recommendations.

APPENDIX A

VISUAL CHECK LIST WITH COMMENTS

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Lake Housatonic Dam

DATE 07/07 & 07/15/81

TIME 10:00 a.m.

WEATHER Sunny, 90°

W.S. ELEV. 25.5

PARTY:	INITIALS:
1. Reynold A. Hokenson	RAH
2. Miron B. Petrovsky	MBP
3. Richard R. Zavesky	RRZ
4. Jerry R. Waugh	JRW
5. Ernst H. Buggisch	EHB

PROJECT FEATURE:	INSPECTED BY:
1. Earthfill Dike	RAH, MBP, EHB
2. Canal Dike	RAH, MBP, RRZ, JRW
3. Left Gatehouse	RAH, MBP, RRZ, JRW
4. Right Gatehouse	RAH, MBP, RRZ,
5. Left Gatehouse	RAH, MBP, RRZ, JRW
6. Right Gatehouse	RAH, MBP, RRZ
7. Left Canal	RAH, MBP, JRW
8. Right Canal	RAH, MBP, RRZ
9. Upper Boat Lock	RAH, EHB, RRZ, JRW
10. Right Canal Spillway	RAH, RRZ, JRW, EHB
11. Spillway Dam	RAH, EHB

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Earthfill Dike NAME: RAH, MBP, EHB

AREA EVALUATED	CONDITION
DIKE EMBANKMENT	
Crest Elevation	39.4 (Average)
Current Pool Elevation	25.50
Maximum Impoundment to Date	32.0
Surface Cracks	None
Pavement Condition	N/A
Movement or Settlement of Crest	Crest varies approximately 1 foot along entire length.
Lateral Movement	None
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	No seepage or evidence of separation.
Indications of Movement of Structural Items on Slopes	Concrete wall on upstream slope has a 2 inch wide crack extending the entire height of the structure.
Trespassing on Slopes	Footpath worn through ground cover. Trees ranging from 3 to 10 inches in diameter. Slopes overgrown by brush.
Sloughing or Erosion	Localized erosion along exposed root system near water surface.
Rock Slope Protection	No riprap observed on dike slopes.
Unusual Movement or Cracking at or near Toes	None
Unusual Embankment or Downstream Seepage	None
Piping or Boils	None

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Earthfill Dike (continued) NAME: RAH, MBP, EHB

AREA EVALUATED	CONDITION
Foundation Drainage Features	Unknown
Toe Drains	Unknown
Instrumentation System	Unknown

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Canal Dike NAME: RAH, MBP, RRZ, JRW

AREA EVALUATED	CONDITION
CANAL DIKE EMBANKMENT	
Crest Elevation	27.3 (Average)
Current Pool Elevation	22.71
Maximum Impoundment to Date	31.80 (Lake Housatonic)
Surface Cracks	See pavement condition.
Pavement Condition	Road surface along crest has cracked and heaved.
Movement or Settlement	Variations in the road surface due to freeze and thaw action.
Lateral Movement	None
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	No seepage or unusual movement.
Indications of Movement of Structural Items on Slopes	N/A
Trespassing on Slopes	Trees ranging from 3 to 14 inches in diameter, and heavily overgrown with brush.
Sloughing or Erosion	None
Rock Slope Protection	River side is completely faced with handplaced stone. Canal side has no protection.
Unusual Movement or Cracking at or near Toes	None
Unusual Embankment or Downstream Seepage	Unknown
Piping or Boils	None

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam

DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Canal Dike (continued)

NAME: RAH, MBP, RRZ, JRW

AREA EVALUATED	CONDITION
CANAL DIKE EMBANKMENT	
Foundation Drainage Features	Unknown
Toe Drains	Unknown
Instrumentation System	Unknown

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam

DATE: 07/07 & 07/15/81

PROJECT FEATURE: Left Gatehouse

NAME: RAH, RRZ, MBP, JRW

AREA EVALUATED	CONDITION
OUTLET WORKS - INTAKE CHANNEL INTAKE STRUCTURE	
a. Approach Channel	Housatonic River
Slope Conditions	
Bottom Conditions	
Rock Slides or Falls	
Log Boom	
b. Intake Structure	
Condition of Concrete	Unknown
Stop Logs and Slots	Unknown

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Gatehouse NAME: RAH, RRZ, MBP, JRW

AREA EVALUATED	CONDITION
OUTLET WORKS - INTAKE CHANNEL INTAKE STRUCTURE	
a. Approach Channel	
Slope Conditions	Wooded and overgrown trees ranging from 2 to 14 inches in diameter.
Bottom Conditions	Unknown
Rock Slides or Falls	None
Log Boom	Concrete on log boom/ice barrier is spalling.
b. Intake Structure	
Condition of Concrete	Unknown
Stop Logs and Slots	Unknown

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Left Gatehouse NAME: RAH, MBP, RRZ, JRW

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - CONTROL TOWER</u>	
a. Structural	
General Condition	Good
Condition of Joints	Bricks appear to be solidly intact.
Spalling	N/A
Visible Reinforcing	N/A
Rusting or Staining of Concrete	None
Any Seepage or Efflorescence	Efflorescence of masonry joints.
Joint Alignment	Good
Unusual Seepage or Leaks in Gate Chamber	Flow through gates, however, unable to determine if this was leakage or if a gate was partially open.
Cracks	None
Rusting or Corrosion of Steel	Unknown
b. Mechanical and Electrical	
Air Vents	Windows and openings are boarded up.
Float Wells	
Crane Hoist	N/A
Elevator	N/A
Gate Operating System	Gates are operated manually or operation assisted by a portable electric motor.

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Left Gatehouse (continued) NAME: RAH, MBP, RRZ, JRW

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	
Service Gates	Condition unknown
Emergency Gates	Unknown
Lightning Protection System	Unknown
Emergency Power System	Unknown
Wiring and Lighting System in gate chamber	Unknown

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Gatehouse NAME: RAH, MBP, RRZ,

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	
a. Structural	
General Condition	Good
Condition of Joints	Bricks appear to be solidly intact.
Spalling	N/A
Visible Reinforcing	N/A
Rusting or Staining of Concrete	None
Any Seepage or Efflorescence	Efflorescence of masonry joints.
Joint Alignment	Good
Unusual Seepage or Leaks in Gate Chamber	Flow through gates, however, unable to determine if this was leakage or if a gate was partially open. Seepage through substructure adjacent to river at 20 to 25 gpm (under head ie, "jet").
Cracks	None
Rusting or Corrosion of Steel	Unknown
b. Mechanical and Electrical	
Air Vents	Windows and openings are boarded up.
Float Wells	
Crane Hoist	N/A
Elevator	N/A
Gate Operating System	Gates are operated manually or operation assisted by a portable electric motor.

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Gatehouse (continued) NAME: RAH, MBP, RRZ,

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	
Service Gates	Condition unknown
Emergency Gates	Unknown
Lightning Protection System	Unknown
Emergency Power System	Unknown
Wiring and Lighting System in gate chamber	Unknown

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Gatehouse NAME: ____________

AREA EVALUATED	CONDITION
OUTLET WORKS - TRANSITION AND CONDUIT	
General Condition of Conduit	N/A
Rust or Staining on Conduit	
Spalling	
Erosion or Cavitation	
Cracking	
Alignment of Monoliths	
Alignment of Joints	
Numbering of Monoliths	

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Left Gatehouse NAME:

AREA EVALUATED	CONDITION
OUTLET WORKS - TRANSITION AND CONDUIT	
General Condition of Conduit	N/A
Rust or Staining on Conduit	
Spalling	
Erosion or Cavitation	
Cracking	
Alignment of Monoliths	
Alignment of Joints	
Numbering of Monoliths	

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Upper Boat Lock NAME: RAH, EHB, RRZ, JRW

AREA EVALUATED	CONDITION
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	
General Condition of Concrete	Good
Rust or Staining	N/A
Spalling	N/A
Erosion or Cavitation	None Visible
Visible Reinforcing	N/A
Any Seepage or Efflorescence	None
Condition at Joints	Good
Drain holes	None visible
Channel	
Loose Rock or Trees Overhanging Channel	None
Condition of Discharge Channel	Vertical masonry wall appears sound; no bulging, leaning, or missing blocks. Note: The manually operated steel gates were in the closed position. Both gates appear operable provided they have not been frozen in place by silt. Some leakage was noted at the gate seals.

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Canal NAME: RAH, RRZ,

AREA EVALUATED	CONDITION
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	
General Condition of Concrete	Note: Canal outlet works consist of two abandoned boat locks. The two wooden gates are in the open position while the third gate, located at the canal, is closed. The third gate is steel and only slight leakage was noted through the gate joints.
Rust or Staining	
Spalling	
Erosion or Cavitation	
Visible Reinforcing	
Any Seepage or Efflorescence	
Condition at Joints	
Drain holes	
Channel	
Loose Rock or Trees Overhanging Channel	Overhanging trees were observed on both canal banks ranging from 2 to 10 inches in diameter.
Condition of Discharge Channel	Canal appears to be silted in. Two parking lots have been constructed in the canal, but culverts were installed to convey water the entire length of the canal. Flow in the canal could not be determined.

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam   DATE: 07/07 & 07/15/81

PROJECT FEATURE: Right Canal Spillway   NAME: RAH, RRZ, JRW, EHB

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a. Approach Channel	West gatehouse and boat lock discharge bay.
General Condition	Good
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	None
Floor of Approach Channel	Condition unknown
b. Weir and Training Walls	
General Condition of Masonry	Good
Rust or Staining	N/A
Spalling	N/A
Any Visible Reinforcing	N/A
Any Seepage or Efflorescence	Seepage noted on vertical downstream face of spillway (nonoverflow section) near canal dike; 15 to 20 gpm (clear).
Drain Holes	
c. Discharge Channel	Housatonic River
General Condition	
Loose Rock Overhanging Channel	
Trees Overhanging Channel	
Floor of Channel	Toe of spillway strewn with loose rock (dissipation).
Other Obstructions	

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Spillway Dam NAME: RAH, EHB

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a. Approach Channel	Housatonic River
General Condition	
Loose Rock Overhanging Channel	
Trees Overhanging Channel	
Floor of Approach Channel	
b. Weir and Training Walls	
General Condition of Masonry	
Rust or Staining	
Spalling	
Any Visible Reinforcing	
Any Seepage or Efflorescence	
Drain Holes	
c. Discharge Channel	Housatonic River
General Condition	Good
Loose Rock Overhanging Channel	
Trees Overhanging Channel	
Floor of Channel	Small island (1,000 ft long and 50 ft wide) located 400 ft from the dam. Boulders were located approximately 50 feet beyond the spillway apron near the left abutment.
Other Obstructions	

PERIODIC INSPECTION CHECK LIST

PROJECT: Lake Housatonic Dam DATE: 07/07 & 07/15/81

PROJECT FEATURE: Left Canal NAME: RAH, MBP, JRW

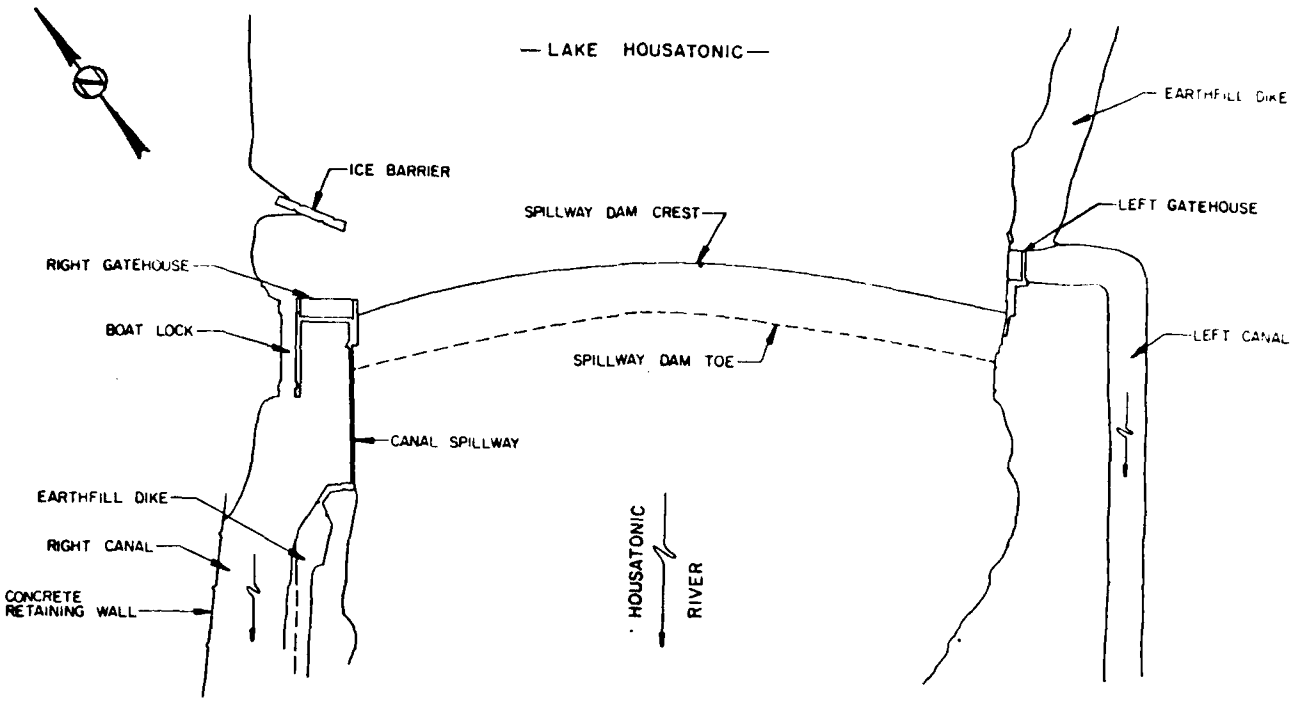
AREA EVALUATED	CONDITION
<u>OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL</u>	
General Condition of Concrete	<u>Note</u>: Canal outlet works (ie. process water intake) near outlet dam have been abandoned. All gate hoist mechanisms (3 are located along the side of the canal near outlet dam, and one operator is located at the outlet dam) appear inoperable. Overall condition of these outlets is unknown.
Rust or Staining	
Spalling	
Erosion or Cavitation	
Visible Reinforcing	
Any Seepage or Efflorescence	
Condition at Joints	
Drain holes	
Channel	
Loose Rock or Trees Overhanging Channel	<u>Note</u>: Small trees along parts the channel. Overhanging trees near outlet dam adjacent to Roosevelt Drive culvert.
Condition of Discharge Channel	Generally clear; however, trees are growing in the channel between the outlet dam and the Roosevelt Drive culvert.

APPENDIX B

ENGINEERING DATA

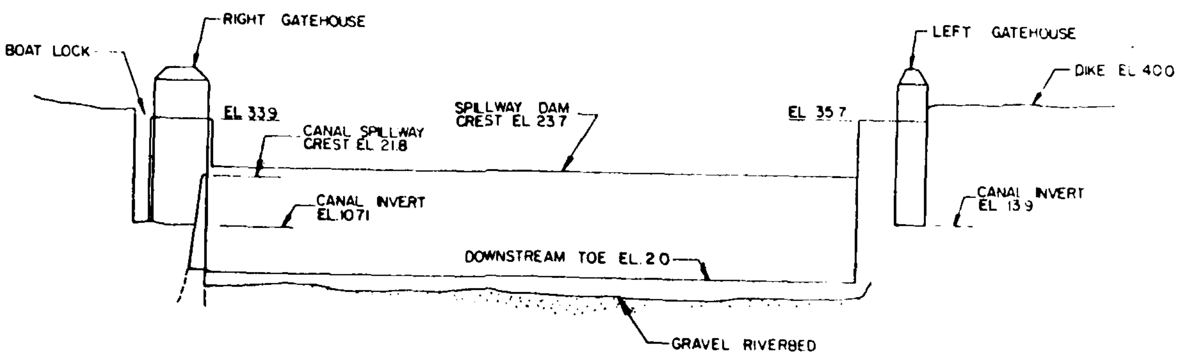
SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
7/81	----	----	Plan, Elevation and Section	B-2
7/81	----	----	Water Resources Inventory Data	B-3
7/15/81		Northeast Utilities Service Company	Piezometer Descriptions and Readings	B-4
----	----	----	Historical Summary	B-8
1979	----	Northeast Utilities Fossil/Hydro Production Dept. Hydro Staff Engineer Ronald Chevalier	History and General Description	B-9
7/81	----	Raul De Brigard	A Short History of the Derby Dam	B-13
12/80	----	RKF	Derby Dam	B-19
1/80	Mr. Robert K. Frink, III Northeast Utilities	Protrans Consultants El Paso, Texas	Stability Analysis	B-24
11/79	----	Northeast Utilities Service Company	Inspection of the toe of the Derby Dam	B-30
3/78	City of Shelton	H.U.D.	Flood Insurance Study (excerpts)	B-31
7/75	V. E. Poeppelmeier	----	Derby Dam Leakage	B-33
11/72	----	Robert N. Smart	Inspection of Derby Dam following excavation downstream of dam.	B-41



PLAN

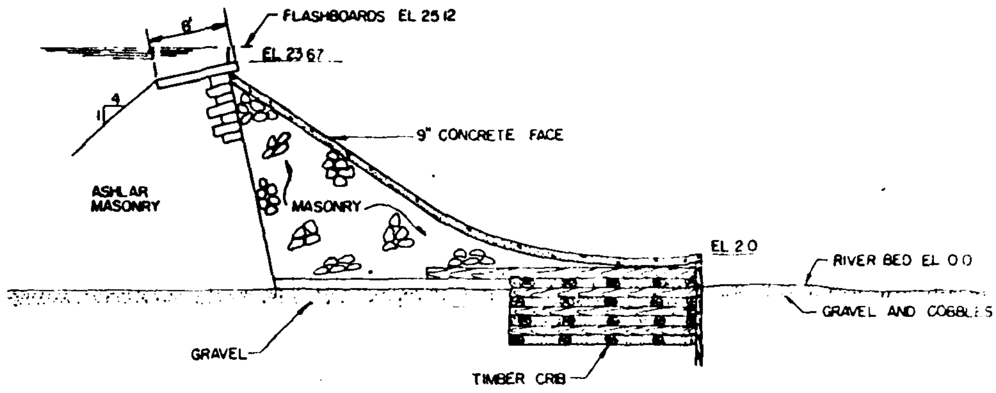
100' 0 100' 200'



PROFILE

HORIZONTAL 0 50' 100' 200' 300'

VERTICAL 0 10' 20' 40' 60'



TYPICAL SPILLWAY DAM SECTION

NOTES:

1. THE SITE PLAN WAS OBTAINED FROM THE PHOTOGRAMMETRIC SURVEY MAPS SUPPLIED BY NORTHEAST UTILITIES SERVICE COMPANY (N.U.), DATED MARCH, 1981.
2. THE DAM SECTION WAS DEVELOPED USING HISTORICAL DATA, A CROSS-SECTION ENTITLED "PLAN OF APRON OF DAM OUSATONIC WATER CO." BY D.S. BRINSMADE, AND THE RESULTS OF THE UPSTREAM SLOPE EXPLORATION PERFORMED BY N.U.
3. THE PROFILE WAS OBTAINED FROM AVAILABLE DATA AND FIELD MEASUREMENTS PERFORMED BY IECO ENGINEERS.
4. ALL ELEVATIONS WERE REFERENCED TO THE PHOTOGRAMMETRIC SITE SURVEY MAP (NGVD).

INTERNATIONAL ENGINEERING CO. DARIEN, CONNECTICUT ENGINEER			US ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	
NATIONAL PROGRAM OF INSPECTION OF NON-FEDERAL DAMS				
PLAN, PROFILE, AND SECTION				
LAKE HOUSATONIC DAM				
HOUSATONIC RIVER			DERBY-SHELTON, CONNECTICUT	
DRAWN BY	CHECKED BY	APPROVED BY	SCALE AS NOTED	
H. Holman	EWB		DATE JULY 1981	SHEET B-2

71

No. ____________

Inventoried By ____________

Date ____________

WATER RESOURCES UNIT
SUPERVISION OF DAMS
INVENTORY DATA

Lat: 41° 19' 19"
Long: 73° 04' 25"

Name of Dam or Pond LAKE HOUSATONIC DAM

Code No. ____________

Nearest Street Location ____________

Town Derby

U.S.G.S. Quad. Ansonia

Name of Stream Housatonic River

Owner Shelton Canal Company

Address 750 Bridgeport Avenue
Shelton, CT (subsidiary of Conn. Light & Power)

Pond Used For Unused   Drainage Area 1574 sq.mi.

Dimensions of Pond: Width ________ Length ________ Area 328 ac.

Total Length of Dam 350'   Length of Spillway 675'

Location of Spillway ____________

Height of Pond Above Stream Bed 23' - 35'

Height of Embankment Above Spillway 4'

Type of Spillway Construction Stone concrete and timber

Type of Dike Construction ____________

Downstream Conditions ____________

900

Summary of File Data ____________

Remarks This is a structure of major importance.

Would Failure Cause Damage? ____________ Class ______

DOWNSTREAM HAZARD? 01

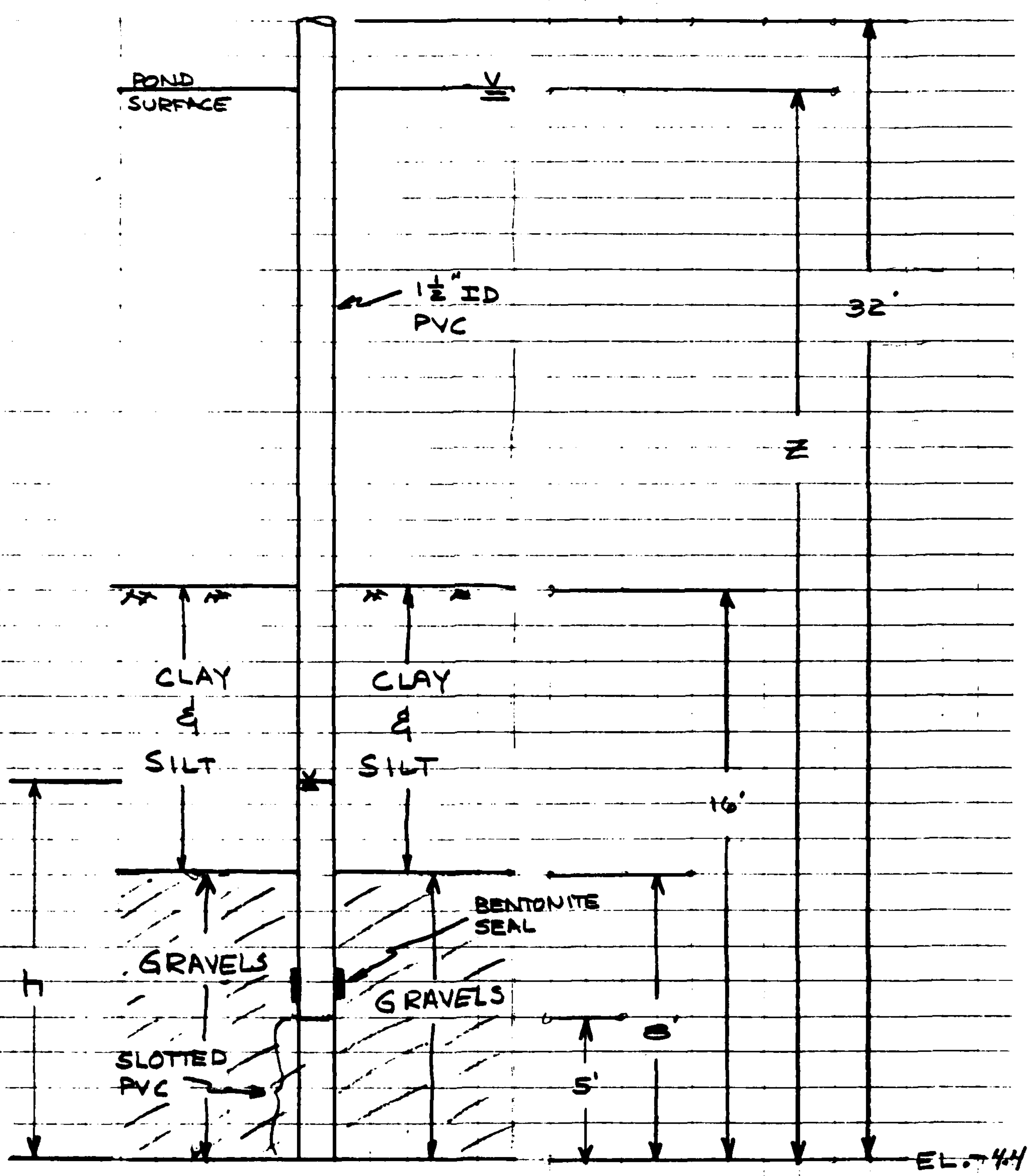
NORTHEAST UTILITIES SERVICE COMPANY

DERBY-SHELTON HYDRO
PIEZOMETERS

SKETCH OF PIEZOMETER #1

FJS DATE 7-15-81

SHEET NO. 1 OF 4

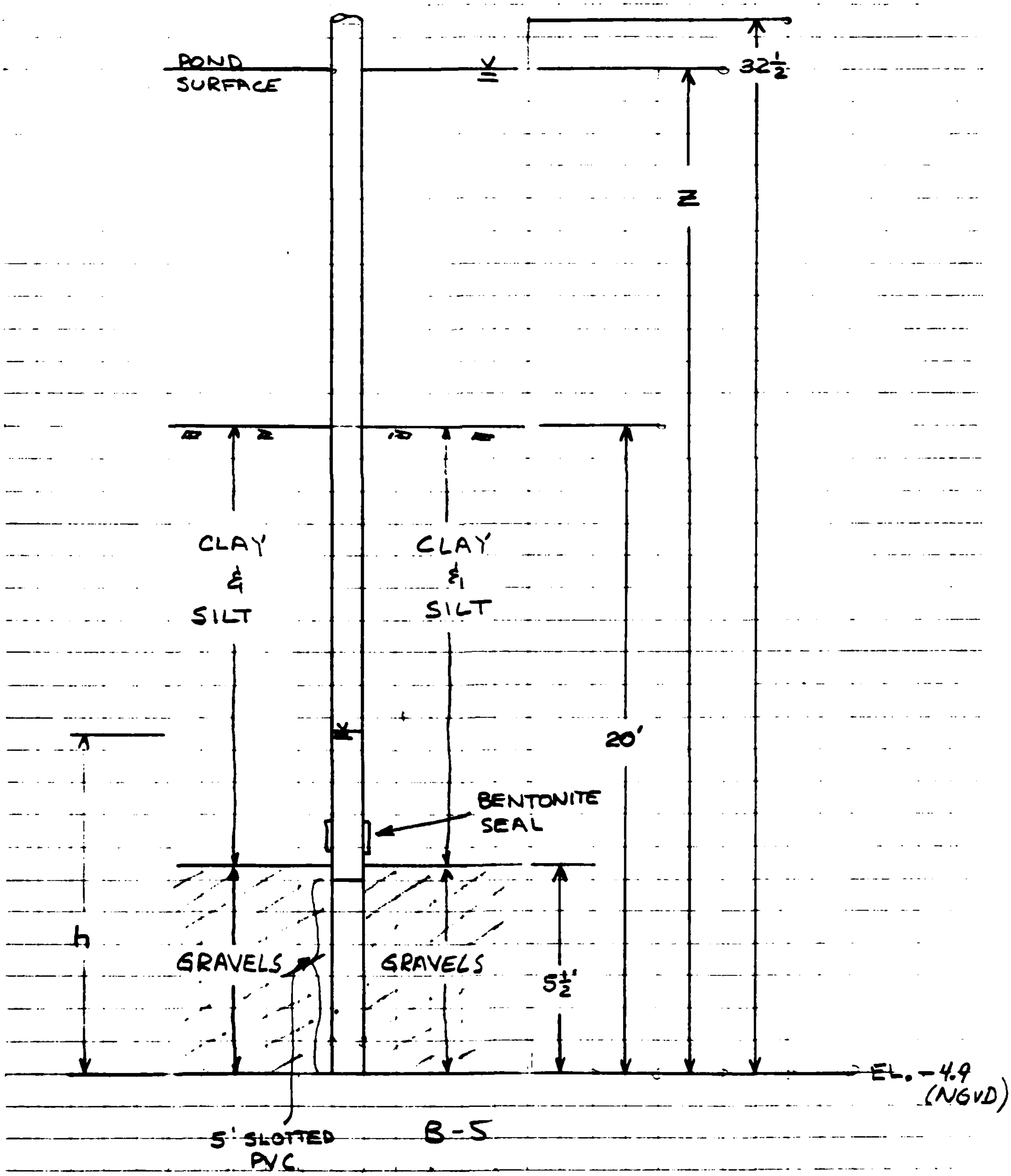


B-4

DERBY-SHELTON HYDRO FJS 7-15-81

PIEZOMETERS

SKETCH OF PIEZOMETER # 2 2 4



NORTHEAST UTILITIES SERVICE COMPANY

DERBY-SHELTON HYDRO — FJS — 7-15-81

PIEZOMETERS

LOG OF READINGS

PIEZOMETER # 1 — SHEET NO. 3 OF 4

DATE	OBSV.	TIME	COND.	[illegible] (FT)	h (FT)	EL. PIEZ.	EL POND	EL TAIL WATER
7-2-81	FW/FJS	8:53A	SUNNY	29.5	9.75	5.35	25.1	0.6
7-2-81	FW/FJS	11:40A	SUNNY	29.5	9.75	5.35	25.1	2.35
7-3-81	FJS	10:50A	RAIN	29.4	9.6	5.2	25.0	0.35
7-4-81	FJS	9:20A	CLOUDY	29.5	10.1	5.7	25.1	0.5
7-5-81	FJS	9:40A	SUNNY	29.75	10.5	6.1	25.35	0.85
7-7-81	FJS	8:30A	"	29.6	11.0	6.6	25.2	0.6
7-8-81	FJS	7:20A	"	29.5	11.4	7.0	25.1	2.3
7-9-81	FJS	8:05A	"	29.4	11.25	6.85	25.0	1.9
7-10-81	FJS	6:20A	"	29.5	11.0	6.6	25.1	2.9
7-11-81	FJS	12:10P	"	29.4	10.7	6.3	25.0	0.6
7-12-81	FJS	5:45P	"	29.6	10.4	6.0	25.2	0.7
7-14-81	FJS	8:05A	"	29.5	10.4	6.0	25.1	0.35
7-15-81	FJS	8:50A	"	29.4	10.5	6.1	25.0	0.35

NORTHEAST UTILITIES SERVICE COMPANY

DERBY-SHELTON HYDRO — FJS — 7-15-81

PIEZOMETERS

LOG OF READINGS

PIEZOMETER # 2 — 4 — 4

DATE	OBSV	TIME	COND	Z (FT)	H (FT)	EL. PIEZ.	EL. POND	EL. TAILWATER
7-8-81	FJS	7:15A	SUNNY	29.5	11.25	6.35	25.1	2.3
7-9-81	FJS	8:05A	"	29.4	10.9	6.0	25.0	1.9
7-10-81	FJS	6:20A	"	29.5	10.7	5.8	25.1	2.9
7-11-81	FJS	12:10P	"	29.4	10.3	5.4	25.0	0.6
7-12-81	FJS	5:45P	"	29.6	10.1	5.2	25.2	0.7
7-14-81	FJS	8:05A	"	29.5	10.0	5.1	25.1	0.35
7-15-81	FJS	8:50A	"	29.4	10.1	5.2	25.0	0.35

SHELTON CANAL AND DERBY DAM

1867 - Construction begins

1875 - Upstream masonry section of dam constructed

1883 - Flashboards first installed

1890 - Dam length increased and rubble fill placed on the downstream side of the masonry to improve structural integrity

1938 - Canal repairs

1939 - Apron repairs

1948 - Reinforcement and concreting of Derby Dam

1952 - Downstream dam face capped with (12) inches of concrete

1952 - Shelton rip rap placement at toe of spillway

1952 - Replacement of highway bridge - Shelton Derby

1956 - Installation of 240' of 72" conduit - Shelton

1956 - Installation of 80' of 72" conduit - Shelton

1958 - Replacement of timber bridge over upper navigation locks of the Shelton Canal

1961 - Installation of 310' of 72" conduit - Shelton Canal

1964 - Roof repairs - Shelton Gatehouse

1964 - Enlarge trashrack

1964-65 - Replacement of Shelton headgate

1965 - Metal stairway to Shelton gatehouse from the spillway overflow

1968 - Replacement of timber bridge over upper navigation locks of the Shelton Canal

1972 - Installation of 72" conduit - Shelton Canal

1979 - Derby Dam toe inspected for undermining (good condition)

1979 - Flashboard repair Derby Dam (done on an annual basis)

SHELTON CANAL COMPANY

I. History and General Description

During the period 1867 to 1870 the Ousatonic Water Company constructed the Derby Dam and two canals - one approximately 2100 ft. long on the Derby side and the other approximately 5400 ft. long on the Shelton side. The Shelton Canal included one upper lock and two lower locks to permit passage of boats.

The objective of the construction was to sell the land between the river and canal for construction of factories. The factories would use the canal for water power and process purposes thus producing income for the water company.

The original water leases covered a period of 90 years bringing the expiration dates to about 1985 to 1995 with the privilege of renewal for a like period. Over the years, and for various reasons, including bankruptcy, most of these old leases have been dropped.

Around 1920 to 1930 the Ousatonic Water Company was purchased by the Shelton Canal Company, a subsidiary of CL&P Co. The purchase was made to secure water rights to ensure the continued operation of the recently constructed Stevenson Station located about 6 miles upstream.

In 1944 there were still a dozen mills using canal water (including the Derby Gas and Electric Co. which ran a steam condenser and two 400KW water wheels).

During the period from purchase of the Ousatonic Water Co. to 1960 the Shelton Canal Co. and Stevenson Hydro were operated and maintained by the CL&P Co. Waterbury District.

In 1960, CL&P Co. purchased the gas properties owned and operated by the Derby Gas and Electric Co. (UI purchased the electric facilities).

CL&P Co., of course, manned the Derby/Shelton area to provide O & M capabilities for its newly purchased gas properties. Because the Shelton Gas Division was a next door neighbor, they inherited the O & M of the Shelton Canal Co. The Hydro Production office assumed responsibility for the Shelton Canal Company in 1979.

II. Condition of Structures and Equipment

The 675 ft. long Derby Dam is in fair to good condition - no major expenditure should be required for 5 years or more.

Both the Shelton and Derby Canals are in poor to fair condition. Canal walls (masonry block construction) require maintenance, fencing is required in many areas for security and safety, equipment is antiquated, and head gates need replacement. Two Shelton lock gates must be replaced if boat passage is to be restored. (Shelton locks have been out of service since 1974).

DERBY DAM
SHELTON AND DERBY CANALS
THE SHELTON CANAL CO.

(1)

(HISTORY AND GENERAL DESCRIPTION (SEE ATTACHED SHEET)

~~[illegible]~~ -

PURPOSE - NAVIGATION AND TO SUPPLY INDUSTRIAL & POWER WATER

DAM - GRAVITY MASONRY

CREST LENGTH 675 FT

HEIGHT 32 FT ± (FOUNDATION TO CREST)

43.5 FT ± (LOW POINT UNDER APRON TO CREST)

LOCATION - HOUSATONIC RIVER (MILE 13.5)

ELEV. CREST = 25.4 FT C.L.&P. DATUM = 23.7 FT. U.S.G.S. DATUM

TOP OF FLASHBOARDS = 26.9 FT. " = 25.2 FT. "

RESERVOIR

AREA (TOP OF F.B.) = 347 ACRES

DRAWDOWN JUNE 30 TO SEPT. 1 = 0.5' (BELOW TOP F.B.) = 173 ACRE FT

NORMAL REST OF YR. = 1.5' (") = 520 " "

MAX. (WHEELS OFF, CONDENSER ON) = 5.5' BELOW TOP F.B.

DRAINAGE AREA = 1575 SQ. MILES (U.S.A.E.)

AVERAGE FLOW = 2680 c.f.s. (32 YRS)

MIN. FLOW = 250 c.f.s.

LOCKS - SHELTON CANAL SIDE

UPPER OR HEAD LOCK 1

LOWER LOCKS 2 ELEV. OF SILLS 2.5' AND 11.6' C.L.&P.C. DATUM

LOCK SECTION OF CANAL 1430 FT ± LONG

BOAT PASSAGES (1951-61 INCL.)

LOW YR. (1951) = 5; HIGH YR (1960) = 15; AVG. YR. = 9

OF 15 IN 1960 ABOUT 50% YALE LAUNCHES (INBOARD 1 @ 28', 3 @ 32'.

OTHERS 12'-18' (OUTBOARDS); NO COMMERCIAL BOATS

ALL COULD USE TRAILERS

RIVER BELOW DAM

EXTREME LOW TIDE 1.5' ± C.L.&P.Co. DATUM

NORMAL LOW TIDE 3.7' ± "

NORMAL HIGH TIDE 5.8' ± "

FLOOD FLOWS

MARCH 12, 1936 H.W. = 33.7'; T.W. = 21.0' C.L.&P.Co. DATUM

SEPT. 22, 1938 H.W. = 33.4'; T.W. = 24.5' " "

AUG. 19, 1955 H.W. = 33.7'; T.W. = 24.9' " "

2

EFFECT OF STEVENSON OPERATION AT DERBY

EACH 2000 KW GENERATION AT STEVENSON = 400 c.f.s.± AT D.

400 c.f.s. AT DERBY FILLS RESERVOIR (LAKE HOUSATONIC) 0.1 FT. PER HOUR WITH NO LOAD AT DERBY

TOTAL LOAD AT DERBY IS ABOUT 1200 c.f.s. OR 0.3' PER HR. IN POND

THIS IS EQUIVALENT TO ~~[illegible]~~ 6000 KW ON ONE UNIT AT STEVENS

FULL LOAD AT STEVENSON (NO SPILLING) AND NO LOAD AT DERBY

= (6120 cfs ÷ 400) × 0.1 = 1.53'/HR

OR = (28800 ÷ 2000) × 0.1 = 1.44'/HR

THIS IS NOT EXCESSIVE

FULL LOAD AT STEVENSON & DERBY (NO SPILLING)

~~[illegible]~~ = [(6120 − 1200) ÷ 400] × 0.1 = 1.23'/HR

AVG FLOW. (NO SPILLING) = (2680 cfs ÷ 400) × 0.1 = 0.67'/HR

MEAN ANNUAL FLOOD AT STEVENSON (BIGWOOD FORMULA)[1] = 20000 cfs

(1) GEOL. SURVEY CIRCULAR 365 - "A FLOOD-FLOW FORMULA FOR CONNECTICUT" BY B.L. BIGWOOD AND M.P. THOMAS

CANALS (AS BUILT)

DERBY SIDE = 2140 FT ±

SHELTON SIDE = 5440 FT ± INCL. 1430 FT. LOCK SECTION (NOTED ABOVE)

ON DERBY SIDE PORTION UNDER CONN. RT. 34 NOW THROUGH ~~[illegible]~~ PIPES

ON SHELTON SIDE LOCK SECTION UNCHANGED.

BELOW LOCK SECTION, BROCK ST. 2-72" PIPES 80' LONG, OPEN CANAL (STAR PIN CO., ONLY WATER WHEEL ON CANAL)

SOUTH OF STAR PIN – 560' ± OF 72" PIPE; 995 FT OF OPEN CANAL TO INTAKE SECTION, SOUTH OF BRIDGE ST.; 971 FT ± 48" PIPE TO SUMP AND INTAKE (FORMERLY S. BLUMENTHAL & CO.); 87 FT ± 24" DRAIN LINE; REMAINDER OF CANAL FILLED IN.

SEE "SHELTON CANAL - 4 SHEETS - SCALE 1" = 40'"

Raul de Brigard 7/81

A SHORT HISTORY OF THE DERBY DAM

The Housatonic River was the last of the major rivers of New England to be dammed at its lower reaches. First came the Merrimac River, at Lowell, in 1826; then the Connecticut River, at Enfield, in 1848; finally, 22 years later, in 1870, the Derby Dam was constructed across the Housatonic.

Compared to its predecessors, the Derby Dam did not include any innovation in its design, other than its slight curve. Its original design was also not especially suited to its location, which made it difficult to build. However, through the preserverance particularly of H. T. Potter, the Superintendent of Construction, and subsequent adaptations and modifications, a stable structure was ultimately achieved that has withstood the test of time.

The notion of damming the lower Housatonic River dates back to 1820. The completion of the first sections of the Erie Canal in that year created a widespread wave of enthusiasm for canals as a superior means of transportation. The thriving commerce, raw materials, iron mines and small manufacturing settlements along the Housatonic River made that river a natural target for canal promoters. In May, 1822, a company by the name of "The Ousatonic Canal" was chartered by the General Assembly in New Haven, within days of similar action of "the Farmington Canal". The two companies became the first chartered canal companies in Connecticut, and the promoters of these companies in both instances used the same Benjamin Wright, Civil Engineer, Chief Engineer of the Erie Canal and leading American authority on canal construction, to make a somewhat hasty but necessary "survey"

required to initially establish the feasibility of the venture. However, while construction was started on the Farmington Canal only five years later, on July 27, 1827, the charter of the Ousatonic Canal expired from its own limitations and an attempt to revive it in 1838 failed again. As explained by Dr. Shelton in 1870, "the requisite legislation was obtained, but as the shad interest was so important, and science has not yet discovered that fish like individuals could climb ladders and go over dams, the company were not permitted to build a high dam like the one (now at Derby), but a low dam, with tumbling rapid over it for the shad. This required the location of the dam near Zoar bridge and the water to be brought down in a canal to the present location or below. The surveys made at the time made the expense so great that it was abandoned".

Renewed interest in damming the river came in 1864. By this time there was no interest in water transportation since the railroad, by 1848, had already made existing Connecticut canals obsolete. Rather, the site of the proposed dam was looked upon as the last available source of industrial water power in close proximity to navigable tide water in all of New England, and the industrial prosperity that resulted from the close of the Civil War gave local promoters and investors the wherewithal to develop this resource to their own advantage. Purchase of the land for the dam and canals began in 1863. An expert was brought from Maine to show a committee of the legislature a model of a fish weir, by means of which fish could go over a high dam. On the basis of this evidence, "the committee were satisfied that they could grant a charter and preserve their respect for the right of the shad and the shad eaters at the same time." Following

this favorable report the Legislature in 1864 granted a charter for the high dam at Derby, notwithstanding the continuing objections of the New Milford shad fishing industry. Funds were obtained in 1866 and the company was organized in November of that year. Plans and specifications were made by Wm. E. Worthen of New York. Henry T. Potter, who had built several dams in the Norwich area (e.g. Ponemah Mills on the Shetucket) was engaged as Engineer and Superintendent. The first stone was laid July 17, 1867.

The construction of the dam was a laborious affair which tried the patience of Henry Potter so much so that after its completion in October, 1870, he declined to have any further involvement with the project. From the description of the construction of this dam given by James Leffel in 1874 it would appear as if, at the time of construction the discovery was made that rock at the site dipped too sharply to be used as a foundation for the dam, a fact that may not have been known earlier by Mr. Worthen when he first made the original plans and specifications for the dam. The laying of a foundation in gravel for a masonry dam, with current from the river above the dam and a three-foot rise and fall of the tide on the downstream side, proved to be a very difficult operation, requiring coffer dams on either side and water pumps to keep the construction area dry. On numerous occasions in 1867, 1868, and 1869 the work was interrupted by freshets breaking through the project's cofferdams. The worst disaster occurred on October 4, 1869, when the center portion of the dam, then under construction, was fully overturned, and 160 feet of the dam was swept away. "The removal of the water from the immense coffer below the dam was a work of such magnitude that the Engineer, Mr. Potter, devised a pump expressly for the purpose, 48 feet long, 4 feet wide, and 12 inches high, with buckets and elevators attached to belts. . . When the water had all

been removed from the coffer, it was found the full extent of the damage done by the October freshet has not been realized. It had not only swept away the center portion of the dam but cut down the riverbed south of the dam, making a hole more than half an acre in extent and 20 feet deep below the apron. This immense cavity was filled with rock and stones, the foundations laid upon it, and on the fifth of October, 1870, the last coping stone was laid." The final structure, estimated to contain 451,000 cubic feet of masonry, was 637 feet long, measure along the arc from abutment to abutment, the arc having a mid-ordinate of 50 feet. The abutments were 175 feet long. The dam was built of large blocks of ashlar masonry. The height varied from 25 feet to 32 feet. Its width was 20 feet at the base, with an 8 foot wide cap of Maine granite blocks each 8 feet long and 1 foot thick. On the downstream side was a horizontal apron, 24 feet wide, of southern pine logs, one foot square, resting on two feet of timber and masonry, with 10 inch sills anchored 8 feet deep into the masonry of the dam. The dam's capability at the time was estimated at 2500 horsepower 12 hours a day, assuming a head of 22 feet and 500 cubic feet per second minimum average flow. On account of the large amount of industrial water power that could be derived from the dam, and the perservance required for its construction, its completion was locally hailed as one of the major achievements of that time. There were no flashboards until ~~1888~~ 1883.

Mr. Potter's success in completing the dam was not destined to last. The dam failed in the spring of 1891, probably due to a large amount of ice which had become piled upon the dam during a freshet at the breaking up of the river. A large breach, estimated to be 210 feet wide, was made at the easterly end of the dam. The repair, which took place that

summer, was under the direction of Engineer D. S. Brimsmade, cost $130,000 and consisted of: (1) lengthening the dam by another 38 feet, to its present 675 feet; (2) reconstructing the breached and new section with a different, substantially wider cross-section, such as to have a sloping back in place of old horizontal apron, and (3) adding a triangular section above the old apron of the remaining portions of the old dam, and increasing the width of the apron from 24 feet to approximately 43 feet-6 inches. The downstream portion of this apron was supported by a rock filled timber cribbing. The toe was protected by 3" plank sheeting just upstream from the last foot-square horizontal waters, which in turn, were laterally supported by piles driven 24" on centers. The entire surface of the apron was then covered with timbers and 2 layers of planking well anchored to masonry, starting just under the capstone and extending at a 1-1/3 to 1 slope, approximately 23 feet and thence through an arc of about 38 feet radius for another 31 feet, to the toe. The new portion apparently built monolithically and of larger stone than the old one, had also vertical upstream face. The chosen method of strengthening the old dam, i.e., by adding a sloping, planked triangular section which would serve at the same time soften the fall of the water, was not original. A very similar approach was used between 1868 and 1870 to strengthen the Holyoke dam. The dam that existed there at the time was a timber crib dam, 1,017 feet long and 30 feet high, that, like the Derby Dam, was built on an erodable base.

There do not appear to have been further modifications to the Derby Dam until 1948. An inspection of the toe of the dam was made in 1943. By 1948, because of the daily operation of the Stevenson Hydroelectric plant which had been constructed in 1920, about six miles upstream, and because of local pond levels from Memorial Day to Labor Day each year, and the extensive worn condition of the subplanking due to erosion, it was found

very difficult and expensive to maintain the repairs on the wooden apron of this dam. That year, under the direction of Hydraulic Engineer D. M. MacWilliam, the Connecticut Light and Power patched with concrete 3,183 square feet of the apron. In addition, on the Derby end of the apron, a 2,290 square-foot experimental strip was laid down, removing both layers of planking and replacing them with 9" of concrete. Additional concrete patches were made in subsequent years. Then, in 1952, C. W. Blakeslee & Sons was awarded the contract for removing all the remaining planking replacing it with nine inches of concrete.

In 1929 the Connecticut legislature released the then Ousatonic Water Power Company of the obligation to maintain a fish weir at the dam. There had been no shad run in the Housatonic River since the turn of the century. Inspectors for the State Fish Commissioners noted shrinking shad runs in the early 1870s. The fish weir as built was not as successful as had originally been anticipated. Samuel Orcutt, in his 1880 History of Derby, describes it as follows: "a weir of fish weir through which an occasional June shad with a sprinkling of youthful lamprey eels are allowed to go up for the special benefit of the up country people".

DERBY DAM

According to the old records, the Derby dam was under construction from 1867 to 1870. While under construction during the flood of October, 1869, a 160 feet of the dam was destroyed and subsequently rebuilt. The grand opening was Saturday, October 29, 1870. Some of the pertinent dimensions of this original dam are as follows: It was 637 feet long, measured along the arc from abutment to abutment, the arc having a mid-ordinate of 50 feet. The abutments were 175 feet long. The dam was of LARGE BLOCKS OF ASHLAR masonry varying in height from 25 feet to 32 feet and capped with granite blocks from Maine. In width it was 20 feet at the bottom and 8 feet at the top. On the downstream side and approximately 20 feet below the crest was a more or less horizontal apron 24 feet in width, made of southern pine timbers resting on 2 feet of timber and masonry. This construction can be clearly seen in the accompanying photographs and section made by ~~W.~~ D. S. Brinsmade. No flashboards were installed until 1883. On January 22, 1891, approximately 210 feet, as scaled from a drawing made by Mr. Brinsmade, was destroyed by flood waters. This was at the eastern or ~~Shelton~~ DERBY end of the dam. The dam was reconstructed during the same year and increased in length to its present 675 feet length, the eastern portion being on a tangent to the existing ARC . At this time the cross-section of the dam was changed materially, the old portion as rebuilt having a triangular section of masonry added above the old apron and the apron increased in width from 24 feet to approximately 43 feet, 6 inches, The downstream portion of this apron being supported by a rock filled timber cribbing. The toe was protected by 3" plank ~~sheeting~~ SHEETING just upstream from the last 12" x 12" horizontal ~~whalers~~ WALERS, which, in turn, were laterally supported by piles driven 24" on centers. The entire surface of the apron was then covered with timbers and planking well anchored to the masonry, starting just under the capstone and extending at a 1-1/3 to 1 slope, approximately 25 feet and ~~went~~ thence through an arc of about 38 feet radius for another 31 feet, to the toe. The new portion was similarly

cc R. deBrigarde, D. Raffel (Photrams)

rebuilt, except that it had a vertical upstream face and the masonry was apparently of larger stone and built monolithically. It is significant to note that both of ~~these~~ washouts mentioned above occurred prior to the installation of the new apron.

From the old drawings and subsequent data, we believe that little if any of this dam rests on bedrock. The average elevation of the crest of the dam as established by a survey made by Mr. Edward W. Richie in May, 1927, is 25.536 feet above the Stevenson Datum of The Connecticut Light & Power Company, which datum is 1.82 feet higher than the U.S.G.S. Datum. From drill holes made at this time, Mr. Richie found that rock was present at elevation + 2.5' at the west end of the dam and dipped sharply towards the east so that at the center of the dam the rock elevation was at about -51.7' while at the eastern end of the dam, ~~no~~ rock was encountered ~~even though one~~ AT drill~~ing~~ hole ~~went to~~ 43 AT ELEV -64.5 feet.

In September 1943 we made an examination of the toe of the dam, including the use of a diver, the report of which is attached hereto. No work was done at that time.

Before estimating the cost of repairs if a break occurs, we believe the following facts should be recognized and considered. The greatest depth of water over the dam on record was during the flood of 1869 when part of the construction was washed out. * At this time 13 feet of water went over the dam. The records for the elevation of the headwater during the flood of January 1891 do not seem to be available. Since the Stevenson Dam was built, however, the greatest floods on record ~~was~~ were in March, 1936 and September 1938, at which time approximately 8.4 feet of water went over the dam, making the headwater elevation about 33.9 feet. During these floods, the tailwater elevations were 21.2 feet and 24.7 feet, respectively.

* This is from an old newspaper record and it's accuracy is not known.

This makes the hydraulic head on the dam only 12.7 feet for the former and 9.2 feet for the latter, or hydraulic characteristics approaching a submerged weir. Any increase in the elevation of the head waters, we believe would cause a corresponding increase in the elevation of the tailwater so that the resulting overturning moment on the dam would become less. Although under these conditions, the downstream slippage might increase, we believe the construction is such that the dam could easily withstand this contingency.

From our experience with dam failures at West Thompson, Mechanicsville, Scotland, Dyer Dam and Leesville, we believe that the vulnerable point at Derby dam is not the dam itself but rather failure around the end of the abutments. We especially think a parallel case can be drawn between Scotland dam and the Derby dam because each is a gravity type dam, built on gravel, with a protecting apron. Both failures at Scotland dam occurred around the end of the east abutment, even though in the 1936 flood, the apron was badly undermined. The railroad cut on the east bank at Scotland, we believe, simulates the same condition as the two canals present at Derby and leads us to conclude that any failure would be at these points. Extending the Derby canal northward several hundred feet, as suggested, we believe would not help the situation but might even aggravate it. A better solution, we believe, would be to extend a wing wall of steel sheet piling or other construction, northerly from the gate house to high ground and at such an elevation that it would keep high flood waters confined to the river channel.

As can be clearly seen in the photograph, the masonry at the east end of the dam is integrally connected to the masonry of the wing wall, running north and south from this point, forming a monolithic structure of considerable depth and adequate strength. Furthermore, we believe, that any ~~stelling~~ scouring action that might take place at this point is at normal or low water elevations rather than during flood conditions and does not constitute a flood hazard. Any disturbance

of the silt BLANKET that has formed at this point either by pouring c
or driving steel sheet piling, we believe, would be more harmful than be:

To fill the two holes just downstream from the toe of the d
require approximately 4,056 cubic yards for the westerly hole and 12,753
yards for the easterly one, or say a total of about 17,000 cubic yards.
as evidenced by the 1936 and 1938 floods, we believe the shape of the ap
such that no serious scouring action takes place at these points during
ditions and, therefore, cannot see that filling them up materially impro
stability of the structure.

Waterbury
C4MacW:S

DERBY DAM   [illegible]   12-24-80
RKF

1 of 1

HISTORICAL

UNDER CONSTRUCTION 1867, 1868, 1869, 1870
160 FT. DESTROYED DURING FLOOD OF OCT. 1869
GRAND OPENING SAT. OCT. 29, 1870

DIMENSIONS
ABUT. TO ABUT. ON ARC 637 FT.
MID. ORD. 50'
BOTH ABUTS. 175'
HEIGHT 25' TO 32'
BOTTOM WIDTH 20'
TOP " 8'
CAPPED WITH GRANITE BLOCKS FROM MAINE
APRON WIDTH 24' OF SOUTHERN PINE AT BASE
RESTING ON 2' OF TIMBER & MASONRY.

NO FLASHBOARDS UNTIL 1883

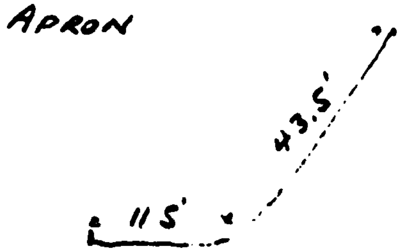
PART OF DAM DESTROYED JAN. 22, 1891   210' ±

RECONSTRUCTED & INCREASE IN LENGTH TO PRES. 675 FT.

RECORD OF FLOODS   Crest Dam = 25.5

YEAR			DEPTH OF WATER OVER DAM IN FT.			Tailwater
OCT. 1869			13			
— 1874			7.8			
SEPT. 1882			6.6			
NOV. 1927			5.4			
NOV. 1932			3.6			
MAR. 1936	[illegible]	8.1	8.4	= EL.	33.9 ±	[illegible]
SEPT. 1938	33.8	8.3	8.4		33.9 ±	24.7

(see [illegible] 2-7-46)



ON WEEKLY BASIS = OCT 1914 = .134

MIN FLOW CONSIDERED TO BE 280 s.f.
0.178 s.f./□MI.

(FROM MEMO OF [illegible] 8-30-40
AVG. DAILY FLOW OF LOWEST MOS. OF LOWEST YR. = NOV. 1914 - [illegible]

D.A. (U.S.A.E.) 1575 □ MI.

CC: R. deBrigarde, D. RAFFEL (PROTRANS)

PROTRANS CONSULTANTS P.O BOX 12608. EL PASO, TEXAS 79912 (915) 532-2282

January 3, 1980

Mr. Robert K. Frink, II
Northeast Utilities Service Company
P.O. Box 270
Hartford, Connecticut 06101

Subject: Hydroelectric-Power at Derby Dam
Your Purchase Order No. 5102320

Dear Mr. Frink:

We are pleased to present this letter-report of our brief reconnaissance of a hydroelectric plant at Derby Dam, on the Housatonic River in Connecticut. Results of our investigations and acknowledgements are given in this initial summary. The studies and our reasoning are then documented in more detail in the remainder of this letter and in the attached Exhibits.

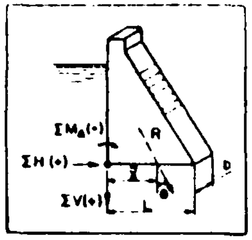
These are our principal conclusions regarding the dam:

1. We expect that Derby Dam will not slide, it should not be washed out, and it should not be overturned, even by the peak flow during a probable maximum flood (PMF). This conclusion is based on stability analyses made by us, for flows up to those causing headwater elevation 38.0 at the dam, and on the report of inspection and on photographs furnished to us by you.

2. We have satisfied ourselves that the PMF developed by others for the Stevenson site upstream is of the correct order of magnitude for use in analyzing the stability of Derby Dam. Independent development of the PMF was beyond our scope of services.

3. In the event our engineering judgement proved wrong and the dam did fail, the potential additional hazard to life or additional damage to property would be negligible or non-existent. If the dam failed, failure would likely be gradual. The rubble fill and concrete debris would block flow through a breach in the dam. The level of water which might flood around the Shelton and Derby sides of the dam would be lower than if the dam did not fail.

4. A minor repair of masonry at the right (Shelby side) abutment should be made in the near future. We do not recommend any other repairs to the dam need be made at this time.



Computed by PEG Date 19 Dec 79 Checked by ______ Date ______ Page ___ of ___

STABILITY ANALYSIS AT ELEVATION -20 msl

Item No.	Force	H	V +	V −	arm	M_A +	M_A −	Remarks
1 to	Granite Blocks							
14	& Rubble Masonry			135,502	38.9	5,272,743		structure
15	16.5×48.6×62.4÷2		25,019		52.4		1,311,010	uplift
16	10.5×48.6×62.4		31,843		44.3		1,910,632	uplift
17	6.5×20×62.4÷2		4,056		13.3		53,945	uplift
18	9.0×20×62.4		11,232		10.0		112,320	uplift
19	9.0×9.0×62.4÷2	2,527			3.0	7,582		tailwater
20	1.5×8.0×62.4			749	51.6	38,638		headwater
21	1.7×8.0×62.4÷2			424	52.9	22,447		headwater
22	3.2×13.1×62.4			2,616	62.1	162,442		headwater
23	23.8×13.1×62.4÷2			9,728	64.3	625,481		headwater
24	(85-62.4)×22.5×22.5÷2	5,721			12.5		71,508	silt
25	1.5×25.5×62.4	2,387			17.8		42,485	headwater
26	25.5×25.5×62.4÷2	20,288			13.6		273,885	headwater
–	5,000×1.0	5,000			30.0		150,000	ice
	Summations	ΣH= 30,869	72,150	149,019		6,130,333	3,4[illegible]	
			ΣV= -76,869			ΣM_A= 2,704,548		

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El Paso, Texas 79912

f = friction factor =
r = shear distribution factor =
A = L × b = area of base under compression =
S_u = ultimate shear stress =
Q = shear-friction factor = $\frac{f(\Sigma V) + rS_uA}{\Sigma H}$ =

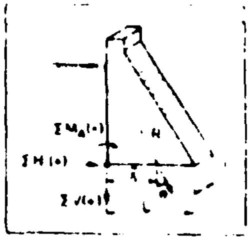
$\bar{X} = \frac{\Sigma M_A}{\Sigma V}$ = -35.2 ft. (upstream) from "A"

inside middle third of base

Pressures
f (toe)
f (heel)

Sliding $\tan\theta = \frac{\Sigma H}{\Sigma V}$ = 0.40

NORTHEAST UTILITIES SERVICE COMPANY
POWER PLANT AT DERBY DAM
STABILITY
NORMAL FLOW & ICE
DEC 28, 1979
Exhibit 2e



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STABILITY ANALYSIS AT ELEVATION -7.0 msl

Item No.	Force	H	V +	V −	arm	M_A +	M_A −	Remarks
1 to 14	Granite Blocks & Rubble Masonry			135,502	38.9	5,273,793		structure
15	16.5×48.6×62.4÷2		25,019		52.4		1,311,010	uplift
16	10.5×48.6×62.4		31,843		44.3		1,410,632	uplift
17	6.5×20×62.4÷2		4,056		13.3		53,945	uplift
18	9.0×20×62.4		11,232		10.0		112,320	uplift
19	9.0×9.0×62.4÷2	2,527			3.0	7,582		tailwater
20	1.5×8.0×62.4			749	51.6	38,658		headwater
21	1.7×8.0×62.4÷2			424	52.9	22,447		headwater
22	3.2×13.1×62.4			2,616	62.1	162,492		headwater
23	23.8×13.1×62.4÷2			9,728	64.3	625,481		headwater
24	(85-62.4)×22.5×22.5÷2	5,721			12.5		71,508	silt
25	1.5×25.5×62.4	2,387			17.8		42,485	headwater
26	25.5×25.5×62.4÷2	20,288			13.5		273,885	headwater
	Summations	ΣH= 25,869	72,150	149,019		[illegible]	3,27[illegible]	
			ΣV = - 76,869			ΣM_A = [illegible]		

f = friction factor =
r = shear distribution factor =
A = L x b = area of base under compression =
S_u = ultimate shear stress
Q = shear friction factor = $\frac{f \Sigma V + r S_u A}{\Sigma H}$ =

$x = \frac{\Sigma M_A}{\Sigma V}$ = -37.1 ft. (upstream) from "A"
Pressure inside middle third
at base
$\frac{\Sigma H}{\Sigma V}$ = 0.34

PROTRANS consultants
P.O. Box 12608
El Paso, Texas 79912

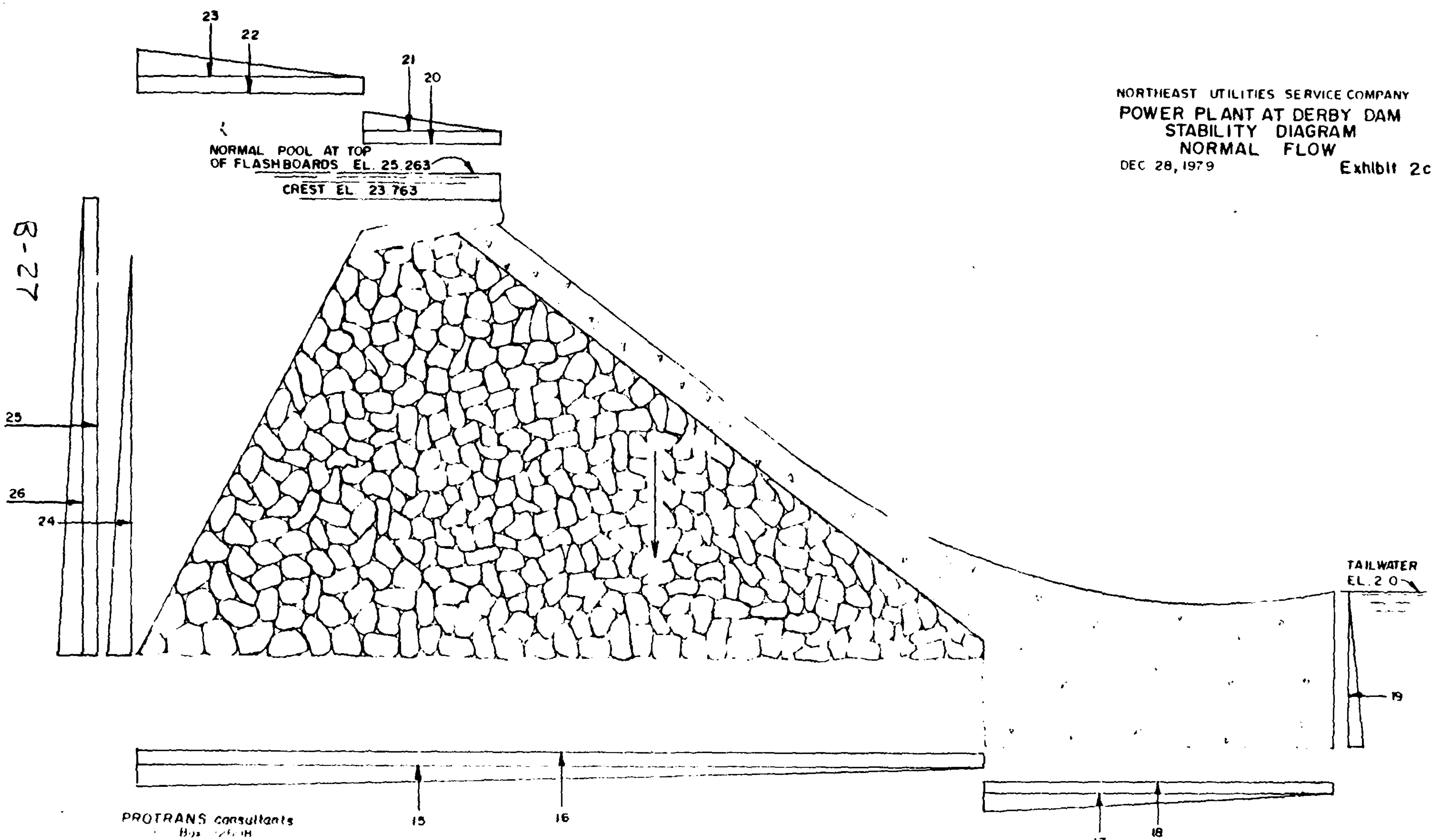
NORTHEAST UTILITIES SERVICE COMPANY
POWER PLANT AT DERBY DAM
STABILITY
NORMAL FLOW
DEC 28, 1979
Exhibit 2d

NORTHEAST UTILITIES SERVICE COMPANY

POWER PLANT AT DERBY DAM
STABILITY DIAGRAM
NORMAL FLOW

DEC 28, 1979

Exhibit 2c



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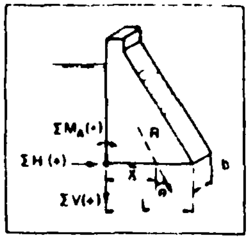


Computed by PEG Date 19 Dec. 79 Checked by ___ Date ___ Page __ of __

STABILITY ANALYSIS AT ELEVATION -7.0 msl

Item No.	Force	H	V +	V −	arm	M_A +	M_A −	Remarks
1 to 14	Granite Blocks & Rubble Masonry			135,502	38.9	5,273,793		structure
15	8.5×48.6×62.4÷2		12,889		52.4		675,369	uplift
16	31.2×48.6×62.4		94,618		44.3		4,191,594	uplift
17	3.5×20×62.4÷2		2,184		13.3		29,047	uplift
18	32.7×20×62.4		40,810		10.0		408,096	uplift
19	2.2×30.5×62.4	4,187			15.2	63,643		tailwater
20	30.5×30.5×62.4÷2	29,024			10.2	296,073		tailwater
21	24.0×14×62.4			20,966	7.0	146,765		tailwater
22	21.8×33.5×62.4÷2			22,785	25.2	574,191		tailwater
23	2.2×33.5×62.4			4,599	30.8	141,696		tailwater
24	14.2×8.0×62.4			7,089	51.6	365,774		headwater
25	1.7×8.0×62.4÷2			424	52.9	22,497		headwater
26	15.9×13.1×62.4			12,997	62.1	807,132		headwater
27	23.8×13.1×62.4÷2			9,728	64.3	625,351		headwater
28	14.2×25.5×62.4	22,595			17.8		402,192	headwater
29	25.5×25.5×62.4÷2	20,288			13.5		273,885	headwater
30	(85-62.4)×22.5×22.5÷2	5,721			12.5		71,508	silt
	Summations	ΣH = 12,393	150,501	214,090		8,316,865	6,051,691	
			ΣV = -63,589			ΣM_A = 2,265,174		

PROTRANS consultants
P O Box 12608
El Paso, Texas 79912

f = friction factor =
r = shear distribution factor
A = L × b = area of base under compression =
S_u = ultimate shear stress

$$Q = \text{shear-friction factor} = \frac{f(\Sigma V) + rS_uA}{\Sigma H} =$$

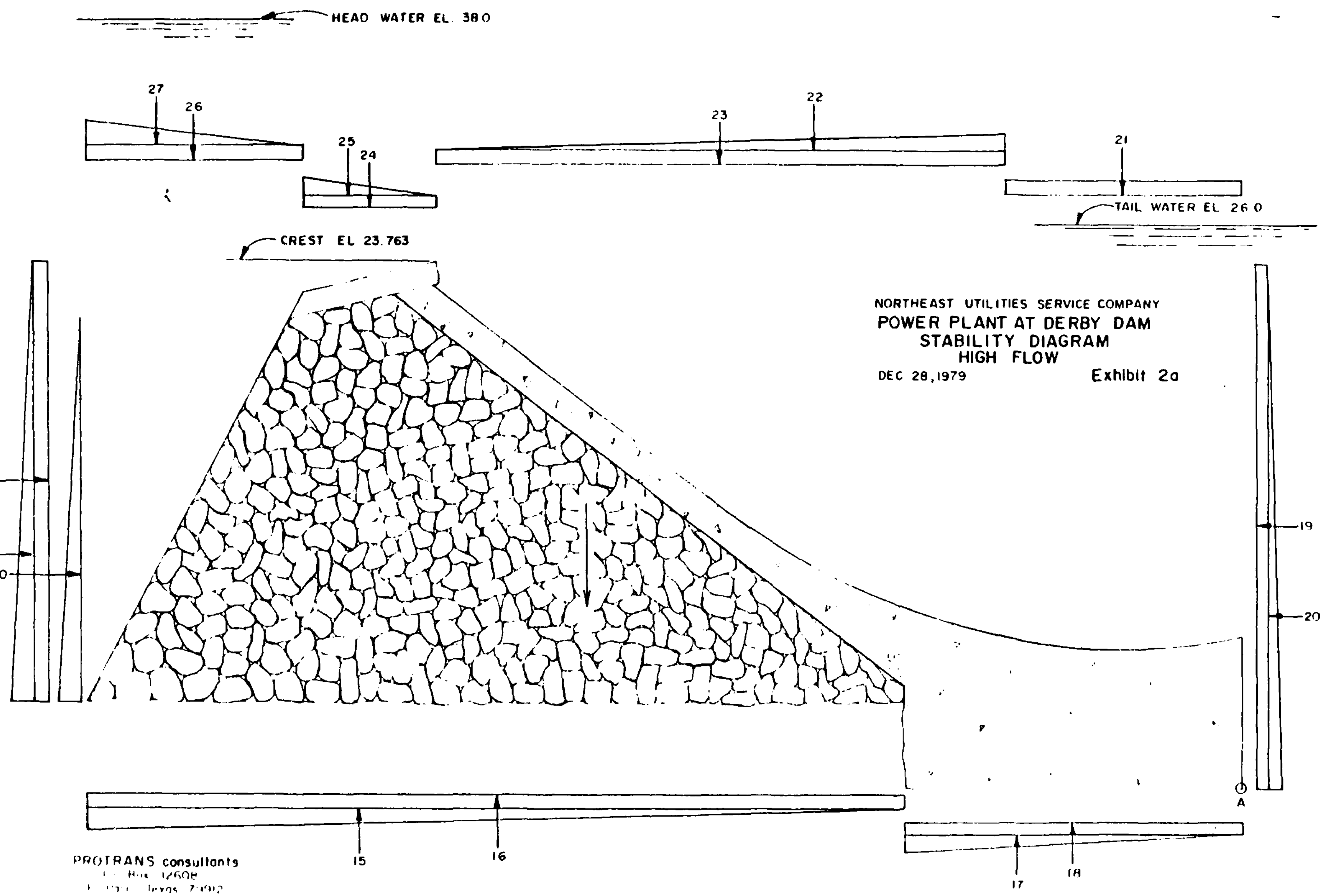
$\bar{x} = \frac{\Sigma M_A}{\Sigma V}$ = -35.6 ft. (upstream) from "A"

Pressures: inside middle third of base
f (toe)
f (heel)

Sliding for A = $\frac{\Sigma H}{\Sigma V}$ = 0.24

NORTHEAST UTILITIES SERVICE COMPANY
POWER PLANT AT DERBY DAM
STABILITY
HIGH FLOW
DEC 28, 1979

Exhibit 26

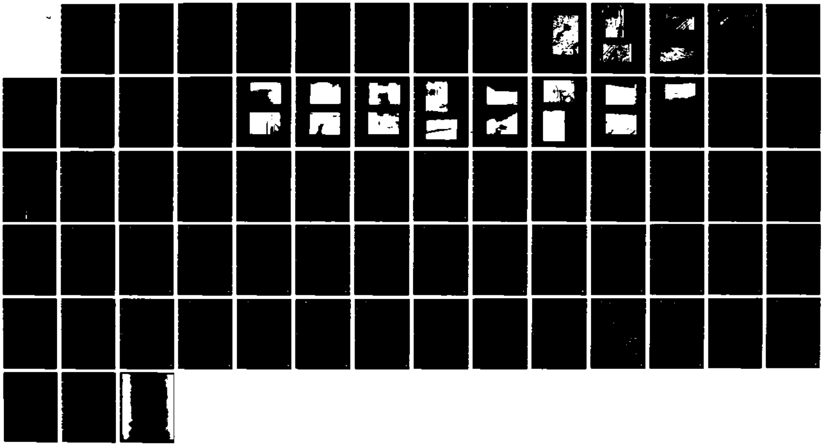


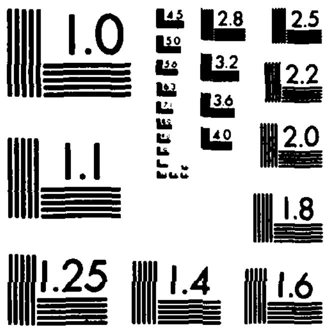
NORTHEAST UTILITIES SERVICE COMPANY
POWER PLANT AT DERBY DAM
STABILITY DIAGRAM
HIGH FLOW
DEC 28, 1979 Exhibit 2a

PROTRANS consultants

AD-A142 592 NATIONAL DAM INSPECTION PROGRAM LAKE HOUSATONIC DAM AND DIKE (CT 00026 AN..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV AUG 81 2/2

UNCLASSIFIED F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

File

DERBY DAM 79

INSPECTION OF THE TCE OF THE DAM ON

SUNDAY, NOVEMBER 18, 1979

INSPECTORS: R. K. Frink
J. W. Ryan
E. W. Zaik

WEATHER: Sunny and warm
55-60°F

CONDITIONS

No flow over the dam, Stevenson ponding, some leakage through and wave action over the flash boards. Time of inspection 1045-1145. The tide was high and ebbing. Face of dam generally visible.

INSPECTION

The toe of the Derby Dam was inspected for undermining. The inspection was carried out by probing with a 10 foot ± wood pole from a row boat. The entire length of the dam was probed. For the middle 475 feet ±, the river bottom at the toe was visible. The bottom consisted of tightly packed cobble size stones. No silt or mud was found. No holes or voids under the toe were found. The face of the dam was visually inspected and it appeared to be in very good condition. The surface was uniformly covered with concrete.

B-30

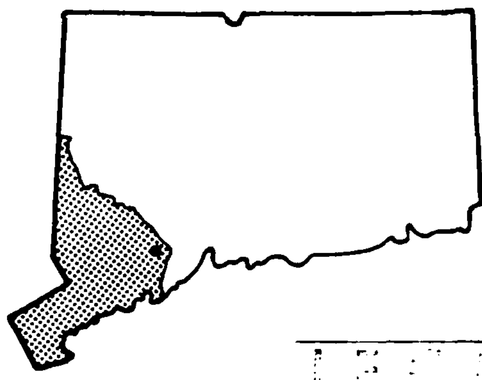
Stevenson
CENTRAL FILE

FLOOD INSURANCE STUDY

1

CITY OF SHELTON, CONNECTICUT

FAIRFIELD COUNTY



MARCH 1978

B-31

U.S. DEPARTMENT of HOUSING & URBAN DEVELOPMENT
FEDERAL INSURANCE ADMINISTRATION

FLOODING SOURCE & LOCATION	DRAINAGE AREA (sq. mi.)	PEAK DISCHARGES (cfs)			
		10-YEAR	50-YEAR	100-YEAR	500-YEAR
FAR MILL RIVER					
Route 110	23.66	4,000	6,300	7,200	9,400
Buddington Road	18.50	3,750	5,900	6,800	8,900
Huntington Road	9.40	1,780	2,760	3,160	4,130
Mohegan Road	6.56	1,540	2,400	2,760	3,600
HARVEY PETE BROOK					
Mohegan Road	2.07	830	1,260	1,430	1,850
Thompson Street	.81	330	490	560	720
MEANS BROOK					
Confluence with Far Mill River	8.59	2,100	3,400	3,900	5,100
Route 108	7.58	1,950	3,170	3,640	4,780
Means Brook Reservoir Dam	5.77	1,710	2,760	3,150	4,120
Route 110	2.68	900	1,350	1,530	1,970
BURYING GROUND BROOK					
Long Hill Avenue	1.43	1,310	1,930	2,180	2,790
Route 8	0.97	750	1,110	1,260	1,610
HOUSATONIC RIVER					
Below Naugatuck River	1,889	55,000	120,000	170,000	220,000
Above Naugatuck River	1,578	45,000	90,000	130,000	198,000

B-32

TABLE 1

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
Federal Insurance Administration

CITY OF SHELTON, CT
(FAIRFIELD CO.)

SUMMARY OF DISCHARGES

FAR MILL RIVER - HARVEY PETE BROOK - MEANS BROOK - BURYING GROUND BROOK - HOUSATONIC RIVER

E. L. Johnson July 11, 1975

V. E. Poeppelmeier

Derby Dam Leakage

As requested, we have observed the leakage on the downstream face of Derby Dam. In order to more clearly understand the situation we include the following brief description of Derby Dam construction:

The upstream masonry section of the dam was constructed about 1875. Around 1890, the dam length was increased and rubble fill was placed on the downstream side of the masonry to improve structural integrity. Finally, in 1952, the downstream face was capped with twelve (12) inches of concrete to reduce maintenance expenses.

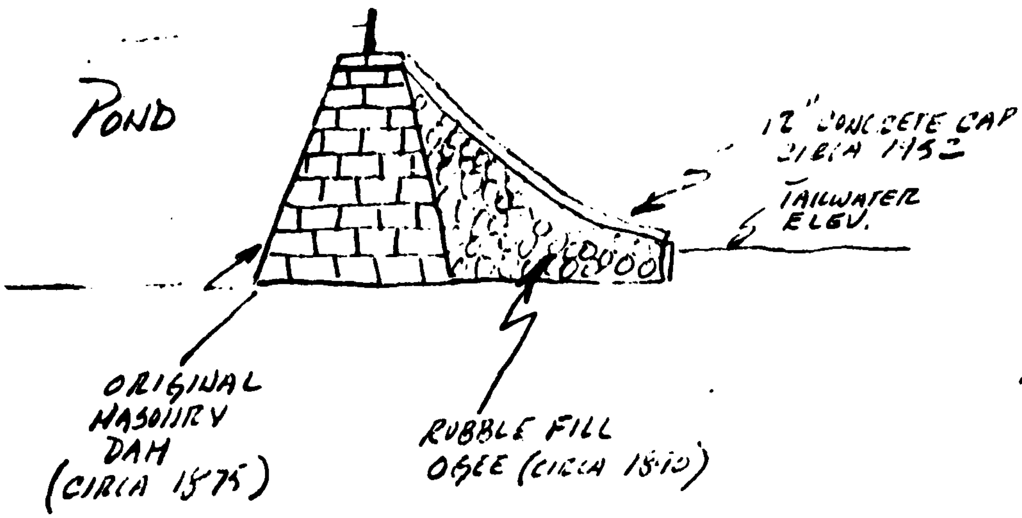
The two leaks visible at the time of inspection were located at the concrete cap construction joints. They were small jets of water, spouting one-half to one inch above the cap, indicating a minimum of water pressure behind the cap. No concrete erosion of significance was visible in the area of the leaks.

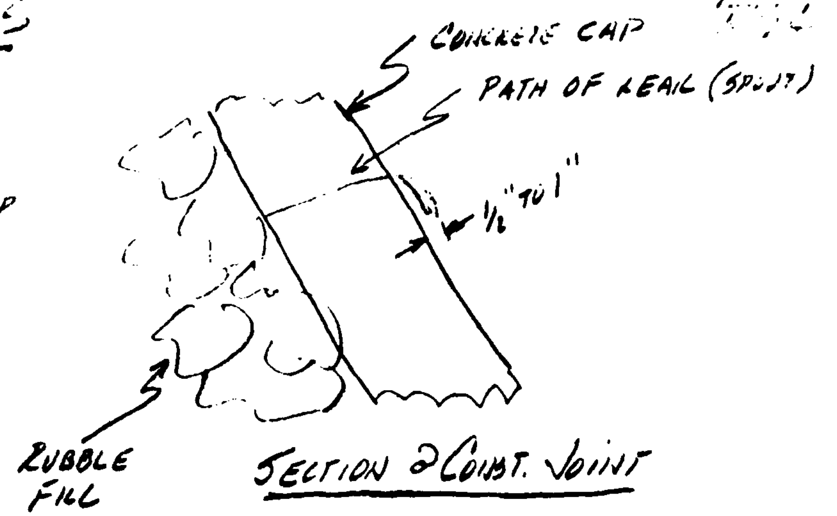
Because of the age of the structure it is doubtful the upstream face is watertight. Some of the water seeping through the upstream face accumulates in the rubble fill between the masonry and concrete cap. Therefore, the leaks through the concrete cap are beneficial as they prevent the buildup of hydraulic pressure behind the cap.

We feel there is no cause for concern at this time. We have scheduled periodic inspections of the dam to accumulate data on the leakage.

RGC/ezb
copy: I.J.Petersen
A. Ferreira
R.G.Chevalier
R.S.Farnum

DERBY DAM LEAKS





SECTION @ CONST. JOINT

THE RUBBLE FILL WAS ORIGINALLY FACED WITH TIMBER TO PROVIDE A RELATIVELY SMOOTH SURFACE. IN 1952, CE+PCO DECIDED TO PLACE A 12" CONCRETE CAP ON THE OGEE TO REDUCE MAINTENANCE COSTS. THE LEAKS R. FARNUM DESCRIBED ARE AT THE CONCRETE CAP CONSTRUCTION JOINTS. TWO WERE VISABLE AT THE TIME OF INSPECTION - THEY WERE SPOUTS OF WATER ABOUT A 1/4" DIAMETER - SPOUTING 1/2" TO 1" ABOVE THE CAP (INDICATING NOT MUCH PRESSURE BEHIND THE CAP). NO CONCRETE EROSION OR [illegible] WAS VISABLE AT THE LEAKS. ACTUALLY, THESE LEAKS ARE BENEFICIAL AS THEY PREVENT THE BUILDUP OF HYDRAULIC PRESSURE UNDER THE CONCRETE CAP. NO REASON FOR CONCERN AT THIS TIME. WILL THIS EXPLANATION SUFFICE OR SHOULD WE PREPARE A MEMO TO ED JOHNSON?

R.G.C.

NORTHEAST UTILITIES
SERVICE COMPANY

July 11, 1975

TO R. G. Chevalier - W210D

FROM A. Ferreira - W46 A. Ferreira

SUBJECT Inspection of Leaks in Derby Dam

The following are comments on the inspection of the reported leaks at the Derby Dam which you requested confirming the discussion we had at the site this morning following the inspection. The leaks were pointed out to us by Bob Farnum, District Superintendent of Gas Operations, Shelton District, who accompanied us to the dam site.

Each of the two leaks that were seen were about pencil size and were spouting approximately one-half inch above the concrete face. The jets were three to four inches in length as they impinged back down on the face of the dam. The jet nearest the southerly abutment was spouting about nine feet below the crest and the other jet was at the next construction joint, spouting about eleven feet below the crest. Each of the jets were at construction joints. There was no evidence of extraordinary concrete spalling or surface wear in the immediate vicinity of the leaks. There was some evidence of concrete erosion closer to the abutment and not at a construction joint. Mr. Farnum was not sure but did think that at an earlier inspection he did see a spouting leak in the vicinity of this eroded area.

The leaks are indicative of water buildup behind the concrete face and are evidence that water is seeping through the masonry dam and through the downstream rubble underlying the concrete face. The spouting leaks are actually relieving the hydraulic pressure buildup under the facing and are, therefore, beneficial in this aspect. It would not be expected that the construction joint would remain watertight and leakage of this type is common.

The safety of this type of dam is not normally impaired by this form of leakage inasmuch there is no impervious fine-grained material involved. Masonry dams, especially of this age and construction, normally exhibit evidence of leakage and seepage through the joints.

The spouting leaks should be continually inspected, however, as their appearance does indicate a possible deterioration of the underlying concrete in the vicinity of the construction joint. There should be no concern unless the leaks increase in jet size and in number.

08-70

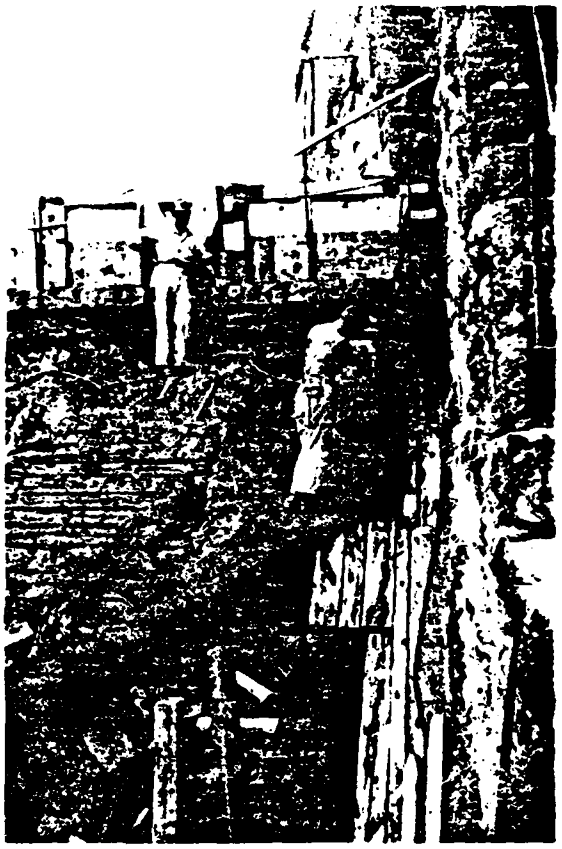
It is recommended that the leaks be monitored periodically and their size and number recorded to build up a progress record.

If there is any further work you want us to do in this regard, please let us know.

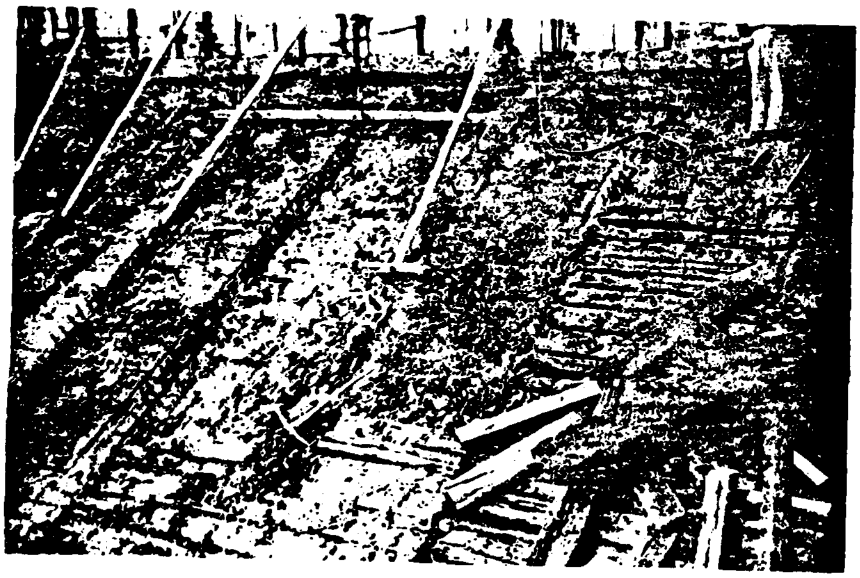
AF/mar

cc: R. S. Farnum
R. P. Werner

Derby Dam June 29, 1939
Apron of Dam at Shelton (West) End before concreting showing leakage through Wicket Gate.



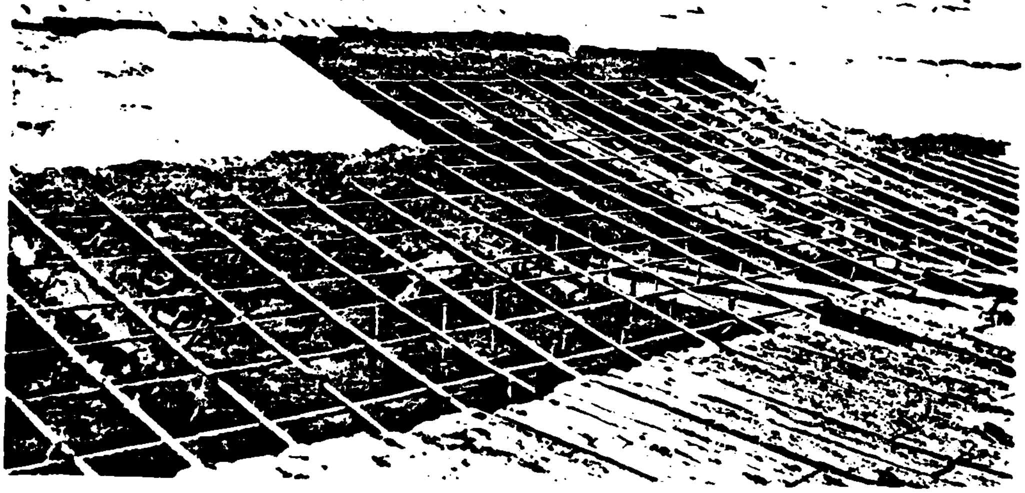
DERBY DAM
SEPT 1948
REMOVING TIMBER
APRON PRIOR TO
POURING CONCRETE



B-38

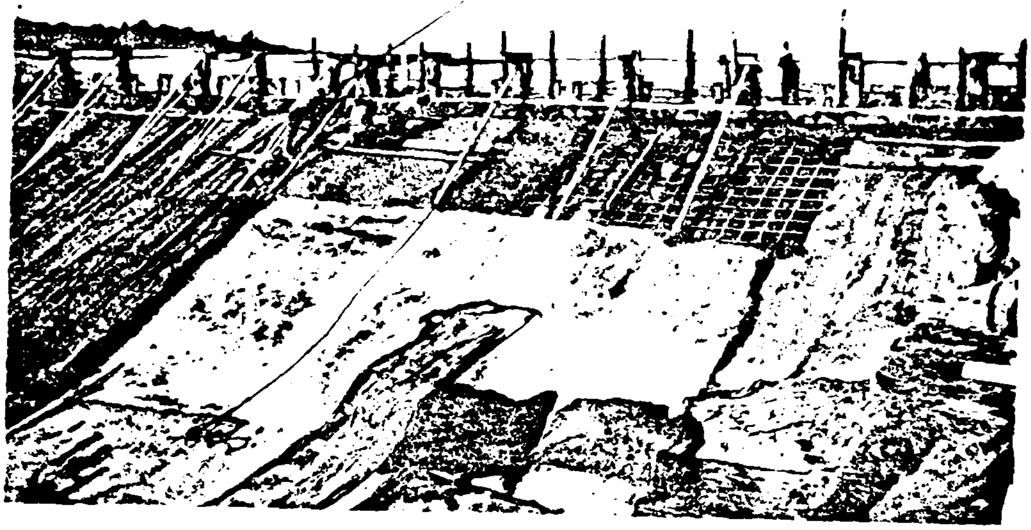
DERBY DAM - OCT 1948
REMOVING TIMBER APRON - 45 FT ON EAST
END OF DAM - PRIOR TO POURING CONCRETE

DERBY DAM - OCT 1948
REMOVING TIMBER APRON PRIOR TO
POURING CONCRETE



DERBY DAM - OCT 1948

APRON REPAIRS SHOWING REINFORCING
PRIOR TO POURING CONCRETE



DERBY DAM OCT 1948
POURING CONCRETE APRON

DERBY DAM

Inspection Following Excavation Downstream of Dam

INTRODUCTION

During the last part of September and the first part of October a borrow pit operation was conducted on an island in the Housatonic River downstream of the Derby Dam, a CL&P property. The excavated material was placed on the easterly edge of the Housatonic River. The contractor was the John J. Brennan Company of Shelton who did the work for the Hull Dye and Print Company. It is understood the Connecticut Department of Environmental Protection issued a stop work order, but not before a large part of the top of the island was stripped.

The possibility the excavation on the island downstream of the dam could adversely affect the stability of the dam has been expressed. A. R. Gierasch and R. N. Smart of NUSCo. accompanied by Elmer Richter of CL&P, made an inspection of the area on October 31, 1971.

SUMMARY

Approximately three to four feet of the top of the island, which was gravel, was excavated. The upstream limit of this excavation was approximately 270 feet downstream of the toe of the dam. Ownership of the island is unknown.

This excavation was not close to the dam and inasmuch as the island was not excavated below the adjacent riverbed elevation, no change in the percolation of water under the dam should be expected. It is our judgement the excavation of the top of the island will not increase the tendency of the flows to scour downstream of the dam.

We conclude the recent excavation has no effect on the stability or structural integrity of the Derby Dam.

SCOPE OF INVESTIGATION

The purpose of the investigation was to inspect the excavation, its proximity to the dam and to ascertain whether the excavation could affect the stability of the dam. This investigation was limited to the effect of the excavation and did not include an inspection and analysis of the dam itself.

DESCRIPTION OF THE DAM

The Derby Dam is a masonry structure 675 feet long varying in height from 25 to 32 feet. Available records indicate the foundation is gravel, the depth to rock varying from two or three feet to 36 feet under the dam and as much as 65 feet just downstream of the dam. The dam was partially destroyed and extensively modified in 1891. Reconstruction included the installation of wood plank sheet piles at the toe of the dam. Eighteen inch high flashboards are attached to the crest of the dam and all of the 675 foot long crest serves as an uncontrolled overflow spillway.

Since the reconstruction several large floods have been passed with no recorded damage to the dam. The largest of these floods was approximately 70,000 cfs on August 20, 1955.

Drawing 36J4, included in Appendix A, is a plan of the dam and riverbed. Important points to note are the two deep holes just downstream of the dam and the island further downstream. It is reported these holes were scoured during the 1891 flood when the dam was partially destroyed. Although we have no survey records, it is reported by the operating staff that the island was deposited, or at least greatly increased in size at this time. Also included in Appendix A are photos taken at 2:00 P.M. October 31, 1972, the river being at a very low elevation with very little flow past the flashboards and the Long Island Sound at low tide.

INSPECTION AND INTERPRETATION OF OBSERVATIONS

Photo one in Appendix A was taken from the Shelton side of the river during the inspection of October 31, 1972, Photo two from the Derby side. The upstream limit of the island excavation shows in photo one and is approximately 270 feet downstream of the dam. Although no surveying was done, it was estimated approximately four feet were excavated from the top of the island. The excavation had been ordered stopped prior to the inspection and a roadway that had been placed between the last bank of the river and the island had been removed. During the period the roadway was in place, the relatively low flows of the river were restricted to the channel on the west side of the island. Now at high tide water passes on both sides of the island.

The section drawings included in the appendix show the relative elevations of the riverbed through the holes downstream of the dam, the elevation of the island and the bottom of the sheet piles, elevation unknown. If one were to attempt to draw flow nets to study seepage under the dam before and after excavation of the top of the island, it would be quickly apparent seepage under the dam will not be significantly changed by the excavation.

The tendency of the riverbed to be scoured downstream of the dam is much more pronounced during high flows. During the 1955 flood the tailwater was approximately El. 24.9 feet, some 25 feet above the general river bottom. The excavation of the top of the island and its subsequent placement at the river's east

bank have little, if any, effect on the cross section of the river at the higher tailwater elevations. Therefore, the tailwater elevations are expected to be unchanged for high flows. It is our judgment that the recent excavation will not make the riverbed more susceptible to scouring than it has been. It should be noted the 1955 flood as well as other large floods since the 1891 modifications have passed the dam without further scouring of the riverbed downstream of the dam.

CONCLUSION

It is concluded the recent excavation of the top of the island can not adversely affect the stability of the dam. The excavation was apparently not on CL&P property. It is our recommendation that no action need be taken by CL&P to restore the top of the island since the stability of the dam will not be affected by the recent excavation.

Robert N. Smart
Robert N. Smart
November 15, 1972

APPENDIX C

PHOTOGRAPHS

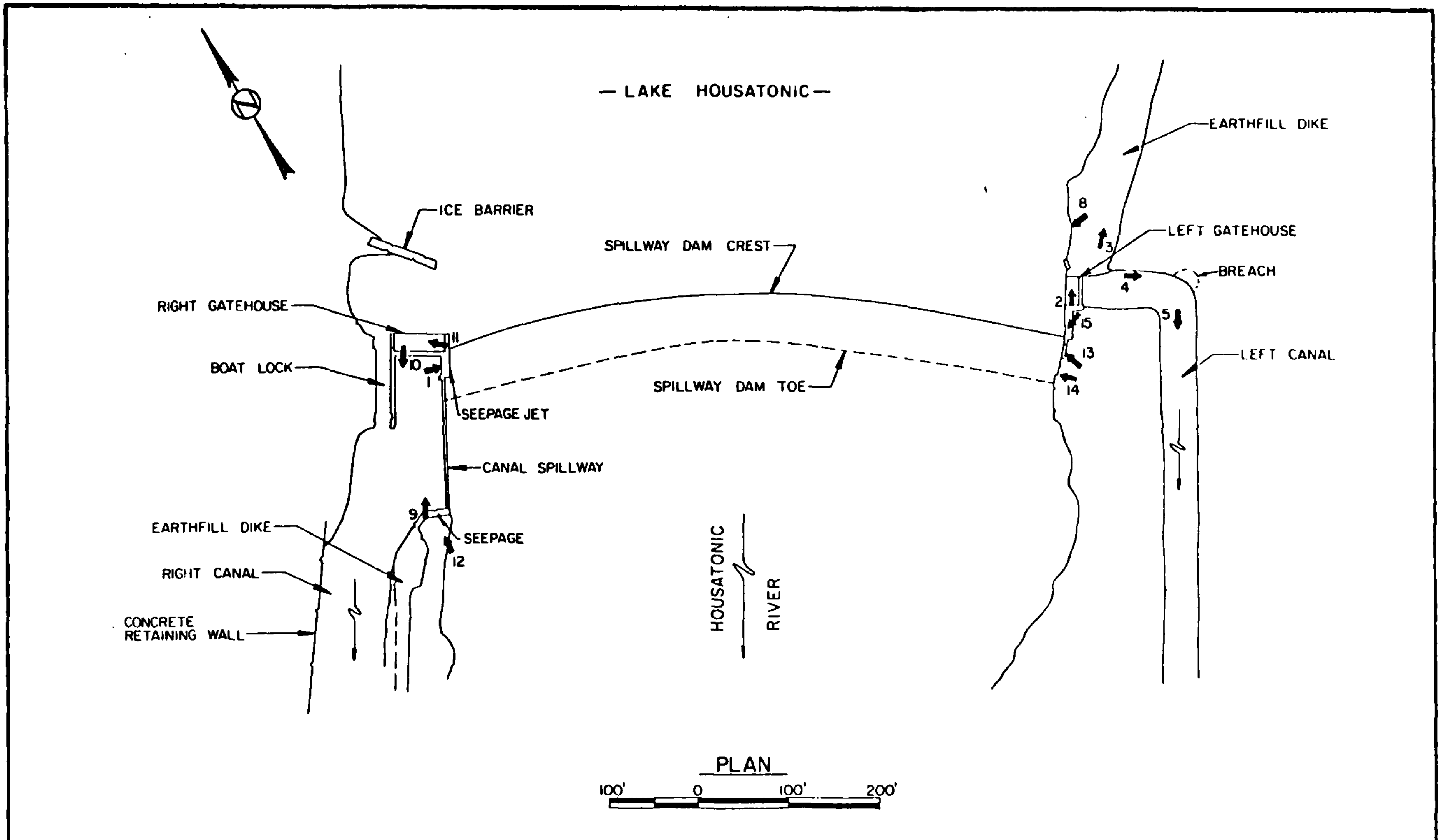


PHOTO LOCATION PLAN

LAKE HOUSATONIC DAM

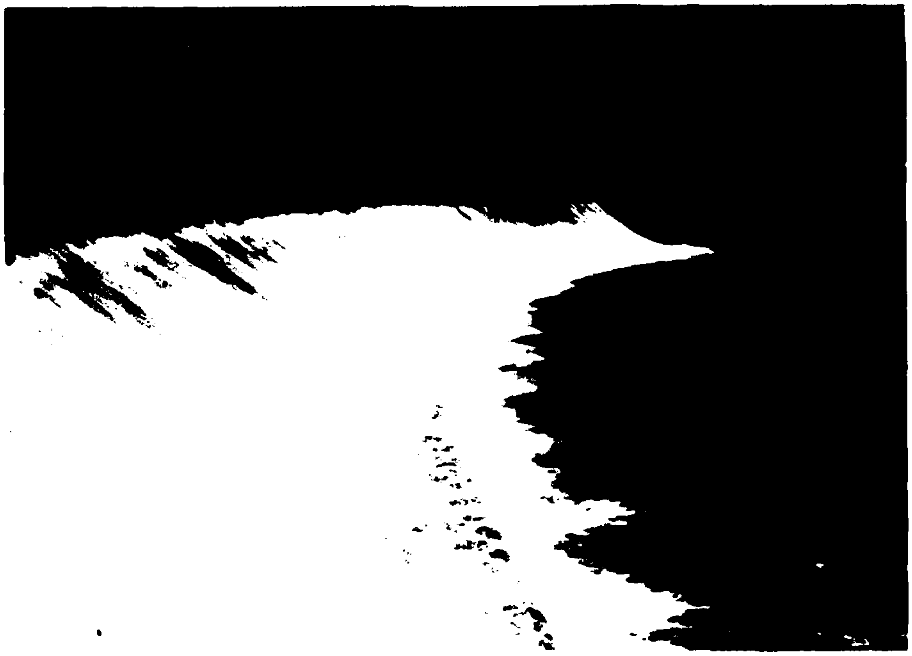


Photo 1 Left Gatehouse and downstream apron of main spillway section.

Photo 2 Interior of Left Gatehouse, gate hoists and portable electric motor for raising gates.

Photo 3 Earthfill dike along left bank extending upstream of dam.

Photo 4 Breach in wall of left canal.

Photo 5 Left Canal along Roosevelt Drive.

Photo 6 Industrial water supply intake works and overflow weir (background) at end of left canal.

Photo 7 Culvert at the end of the left canal under Roosevelt Drive.

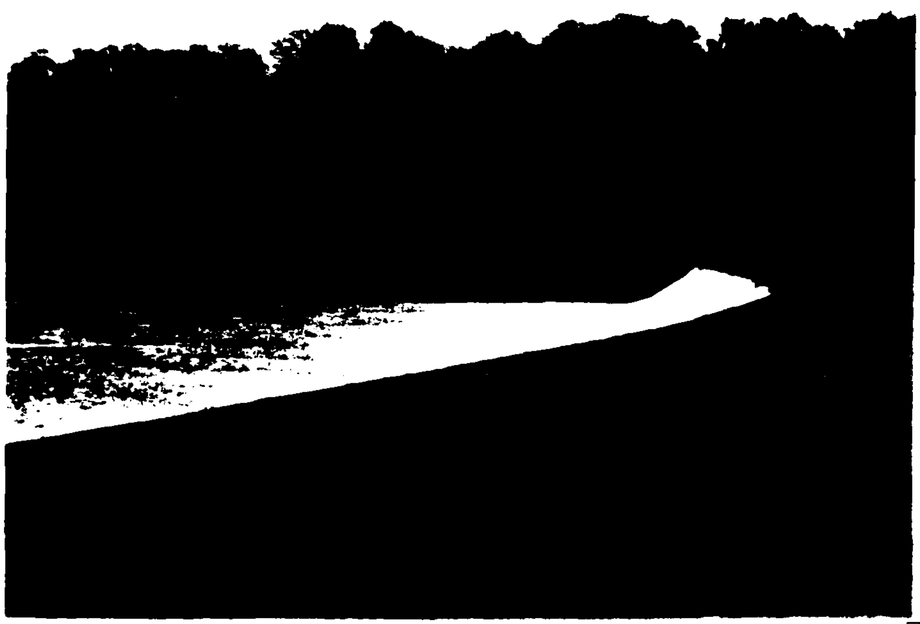


Photo 8 Right Gatehouse and Canal Spillway

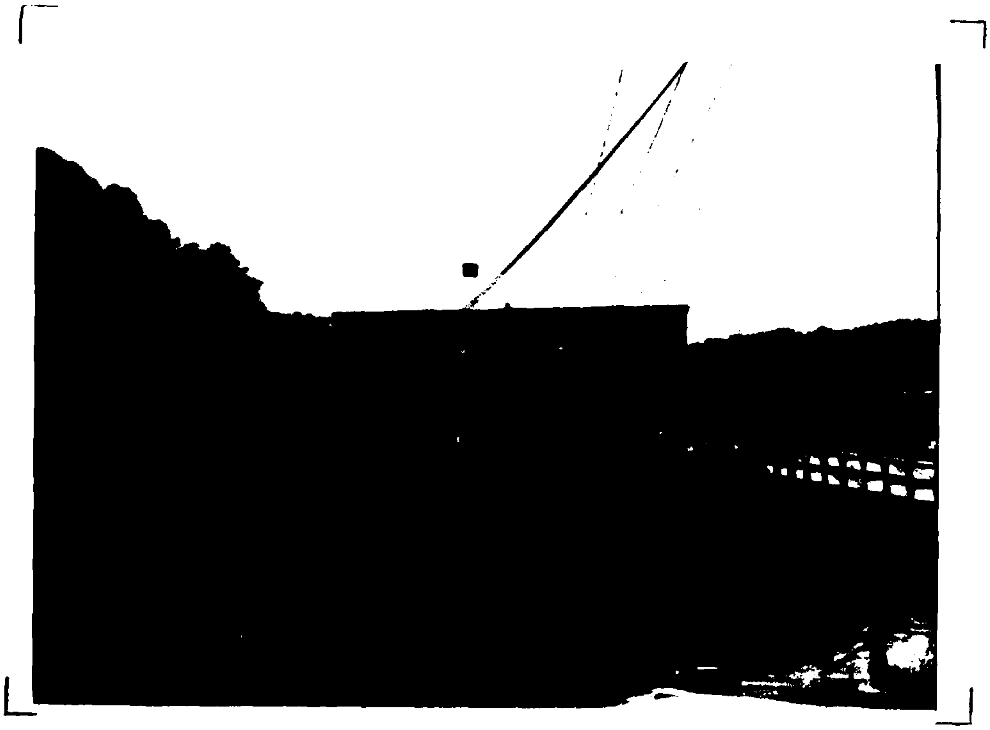


Photo 9 Right Gatehouse; boat lock; and canal spillway.

Photo 10 Right canal; concrete training wall along right bank; and earthfill dike with paved crest on left.

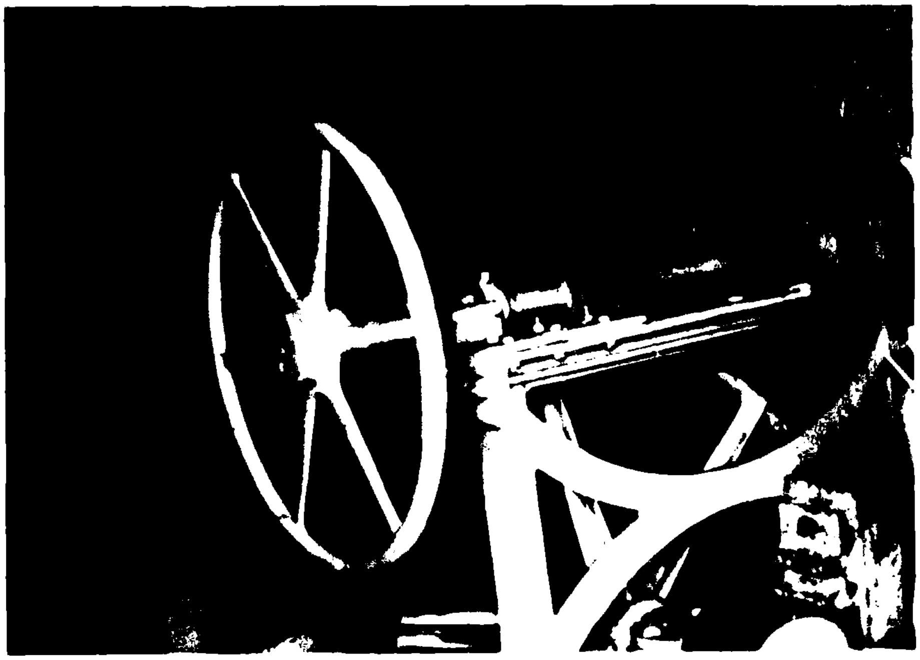


Photo 11 Interior of right gatehouse and gate hoist mechanism. Note belt driven alternate gate hoist system.

Photo 12 Seepage on downstream wall of canal spillway structure. Note stone facing on earthfill dike (left of seepage).

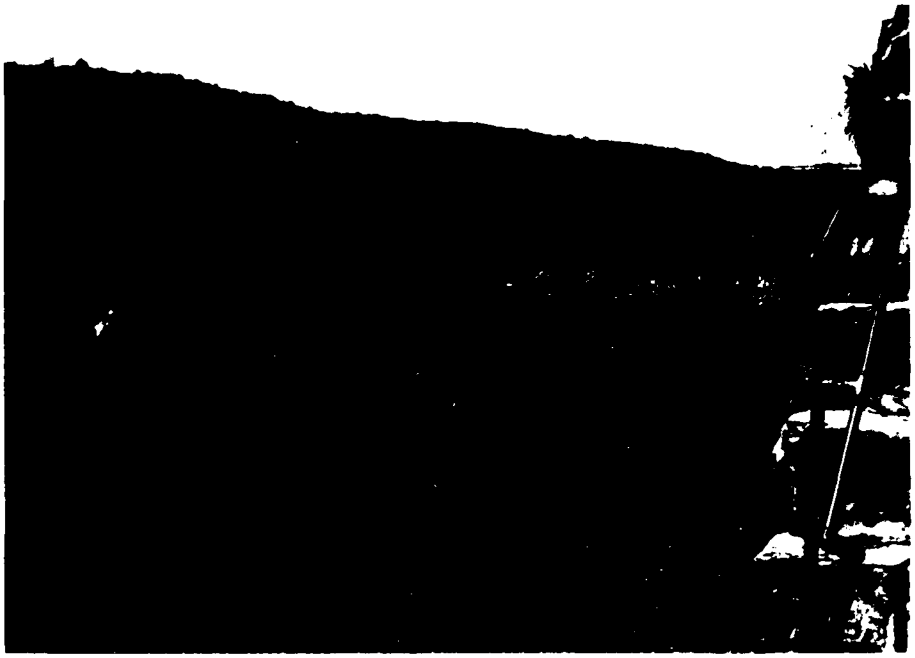


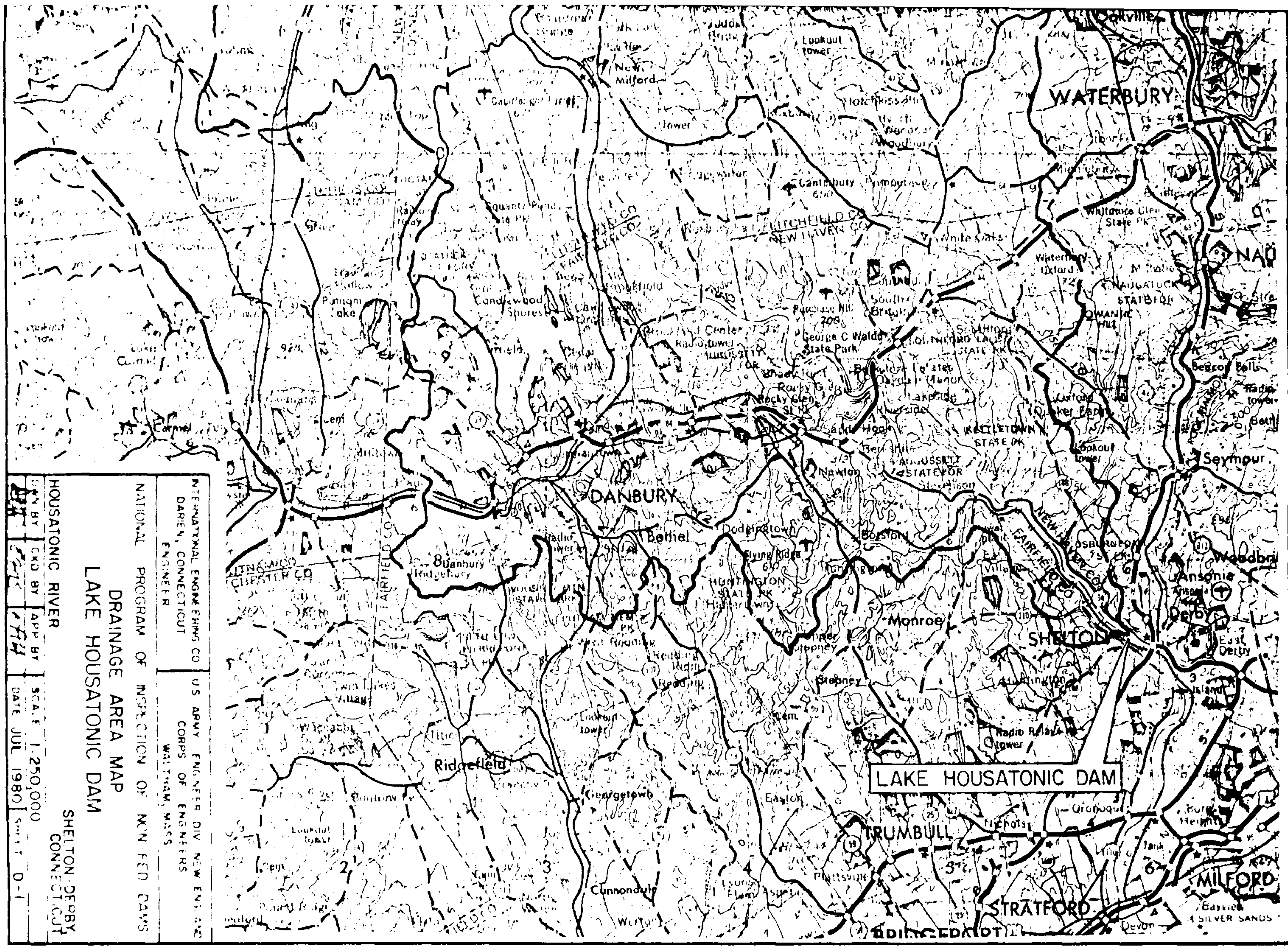
Photo 13 Concrete downstream spillway apron and flashboards.

Photo 14 Spillway apron; right gatehouse; and canal spillway.

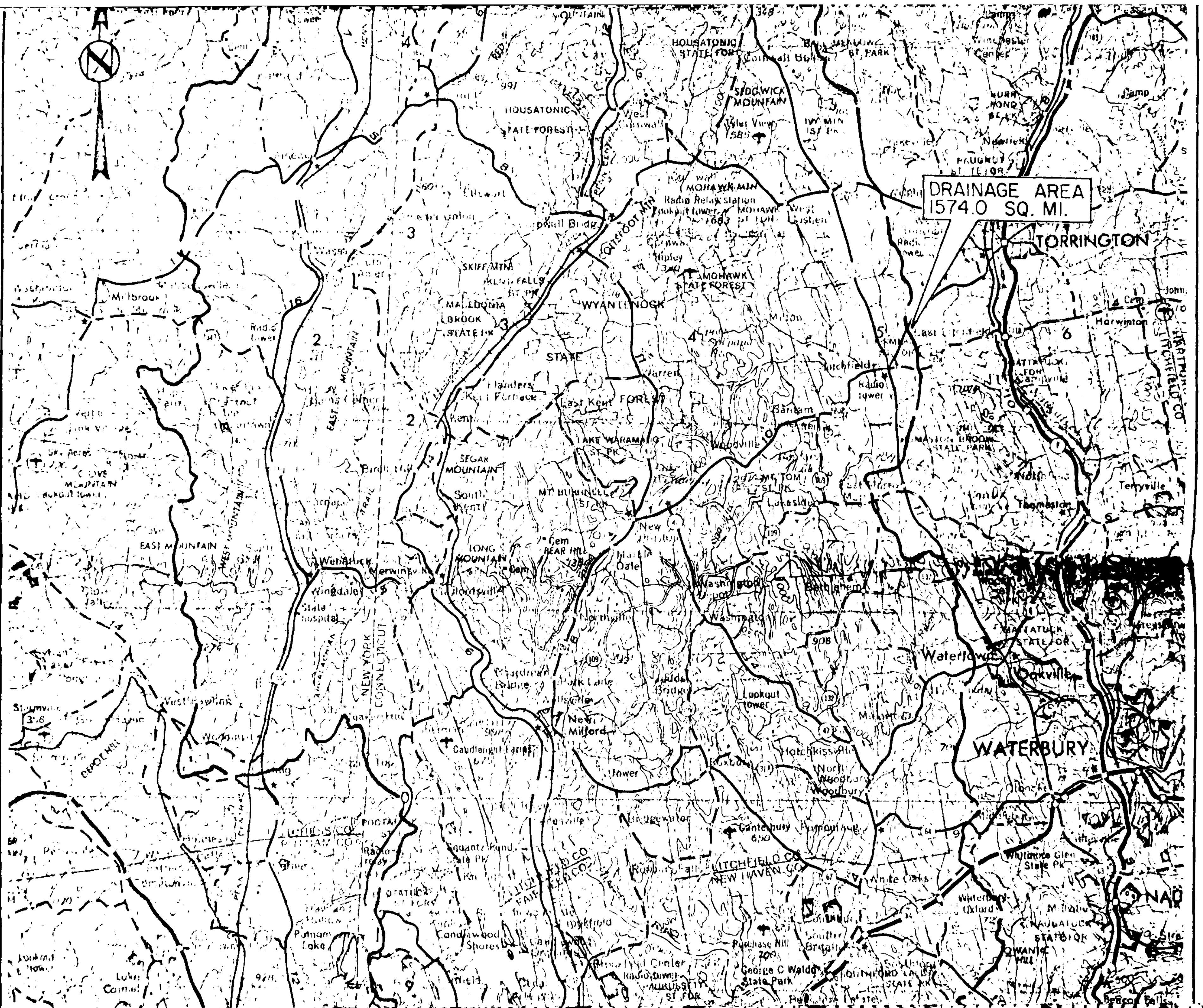
Photo 15 Downstream channel - Housatonic River.

APPENDIX D

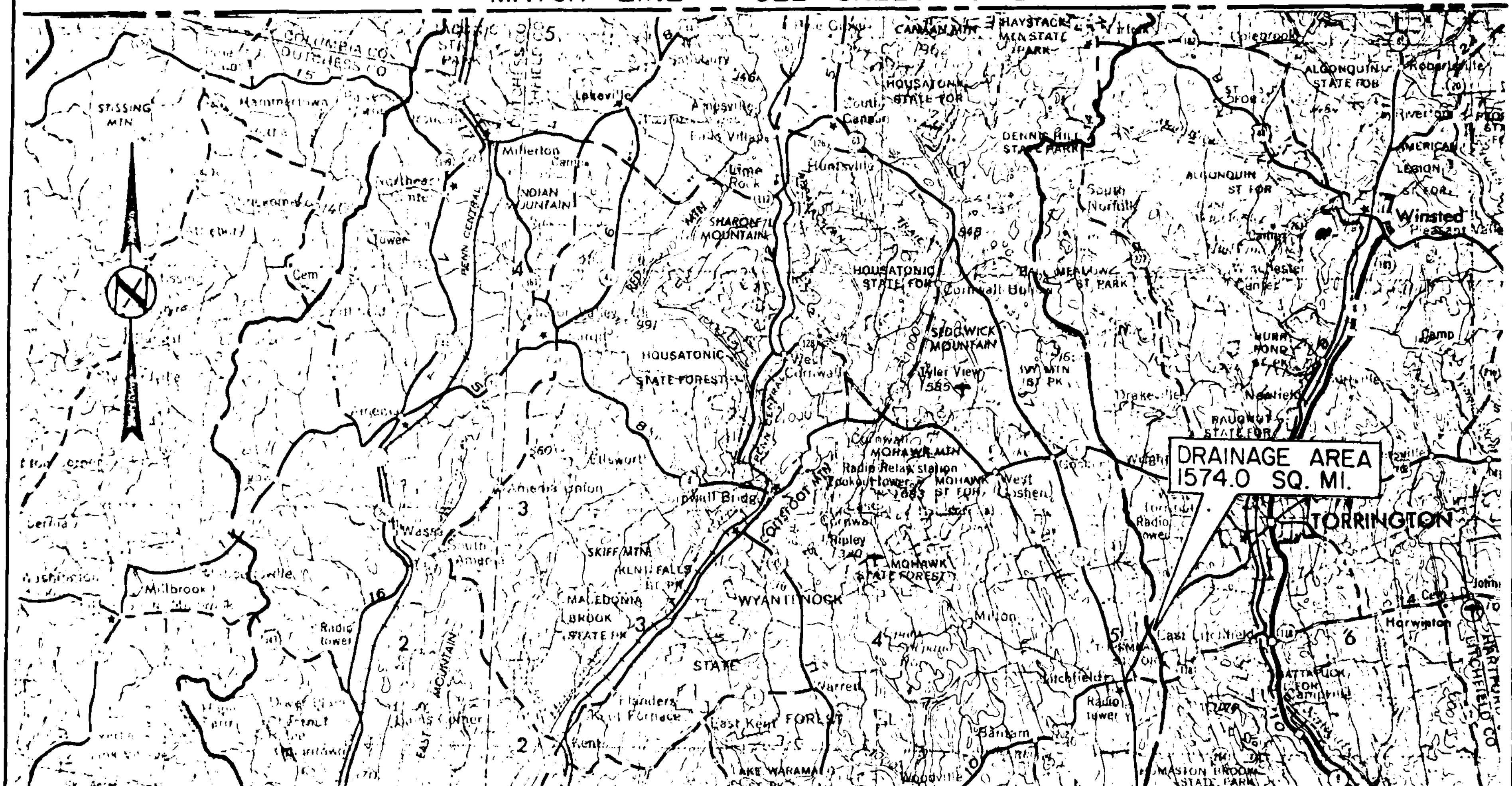
HYDROLOGIC AND HYDRAULIC COMPUTATIONS

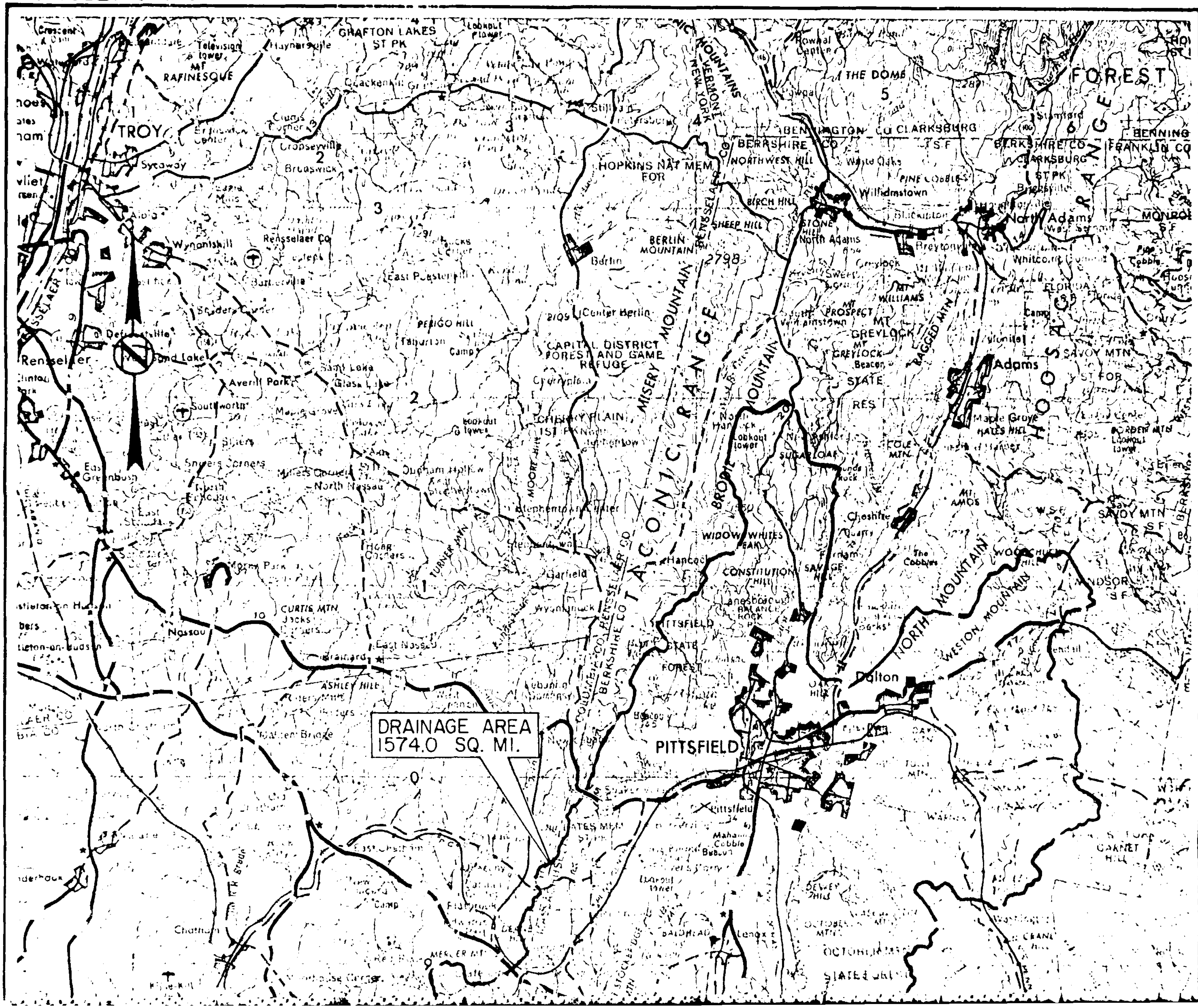


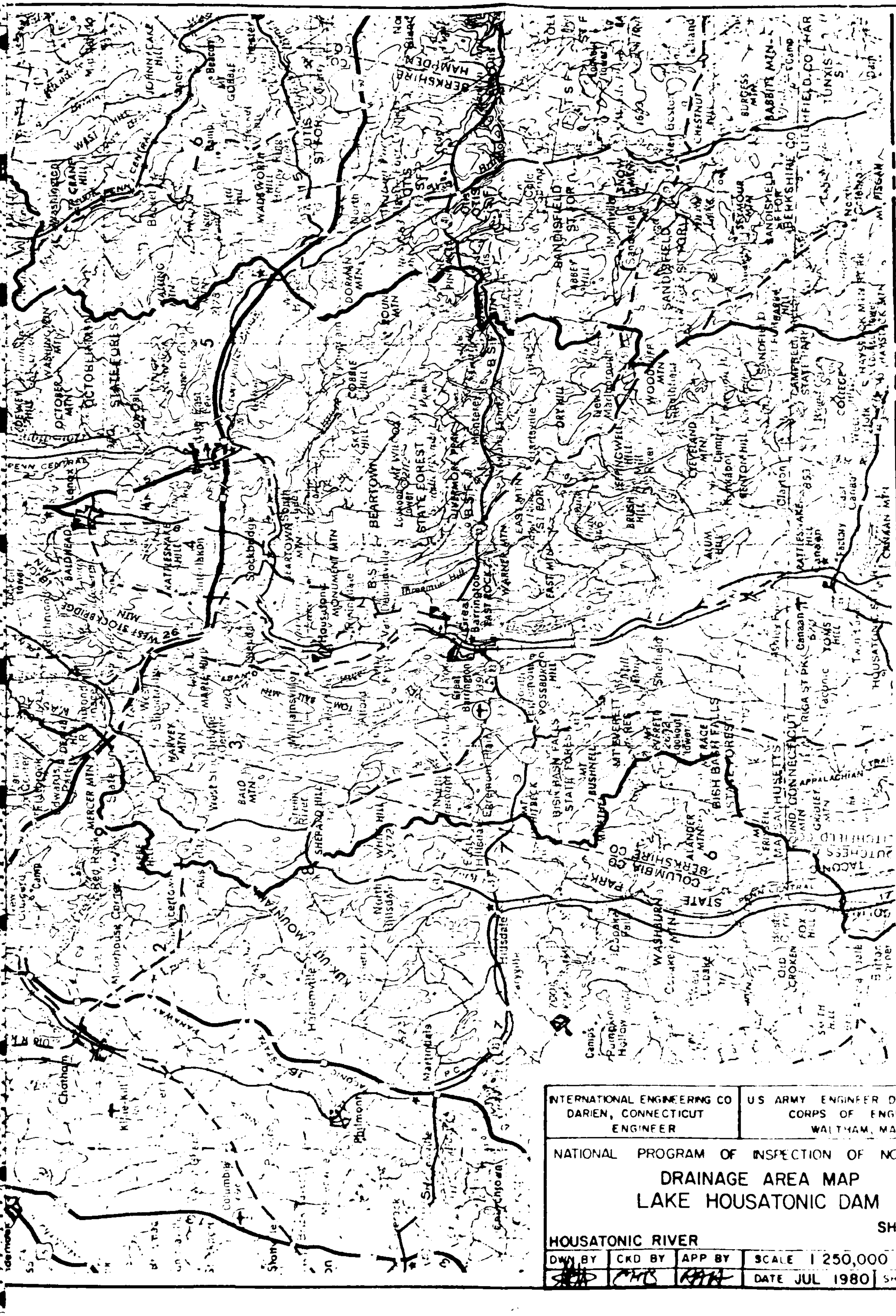
INTERNATIONAL ENGINEERING CO
DARIEN, CONNECTICUT
ENGINEER
US ARMY ENGINEER DIV NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS
NATIONAL PROGRAM OF INSPECTION OF NON FED DAMS
DRAINAGE AREA MAP
LAKE HOUSATONIC DAM
SHELTON-DERBY
CONNECTICUT
HOUSATONIC RIVER
DWN BY
CKD BY
APP BY
SCALE 1:250,000
DATE JUL 1980
SHEET D-1
LAKE HOUSATONIC DAM
WATERBURY
DANBURY
Bethel
SHELTON
Derby
Ansonia
Seymour
Monroe
Ridgefield
TRUMBULL
STRATFORD
MILFORD
New Milford
Newtown
Easton
Stepney
Dodgingtown
Oxford
Southbury
Woodbury
Brookfield
Candlewood Shores
Putnam Lake
Carmel
Georgetown
Cannondale
FAIRFIELD CO
NEW HAVEN CO
LITCHFIELD CO
East Derby
Devon
SILVER SANDS


DRAINAGE AREA
1574.0 SQ. MI.
TORRINGTON
WATERBURY
NEW YORK
CONNECTICUT
HOUSATONIC STATE FOREST
SEDGWICK MOUNTAIN
MOHAWK MTN
MOHAWK STATE FOREST
WYANTENOCK
SKIFF MTN
MACEDONIA BROOK STATE PK
EAST MOUNTAIN
WEST MOUNTAIN
SEGAR MOUNTAIN
LONG MOUNTAIN
LAKE WARAMAUG ST PK
Millbrook
Wingdale
New Milford
Waterbury
Oakville
Harwinton
Terryville
Thomaston
Litchfield
Woodbury
LITCHFIELD CO
NEW HAVEN CO
FAIRFIELD CO
HARTFORD CO
DEPOT HILL
APPALACHIAN TRAIL

MATCH LINE — SEE SHEET D-2




DRAINAGE AREA
1574.0 SQ. MI.
PITTSFIELD
TROY
TACONIC RANGE
HOOSAC RANGE
MISERY MOUNTAIN
BRODIE MOUNTAIN
NORTH MOUNTAIN
WESTON MOUNTAIN
BERLIN MOUNTAIN
2798
MT GREYLOCK
GREYLOCK STATE RES
CAPITAL DISTRICT FOREST AND GAME REFUGE
HOPKINS NAT MEM FOR
PITTSFIELD STATE FOREST
GRAFTON LAKES ST PK
MT RAFINESQUE
THE DOME
FOREST
VERMONT
NEW YORK
BERKSHIRE CO
RENSSELAER CO
COLUMBIA CO
BENNINGTON CO
CLARKSBURG
North Adams
Adams
Williamstown
Dalton
Nassau
Rensselaer
Wynantskill
Averill Park
Berlin
Cheshire
SAVOY MTN
Lenox
DRAINAGE AREA
1574.0 SQ. MI.



INTERNATIONAL ENGINEERING CO DARIEN, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS

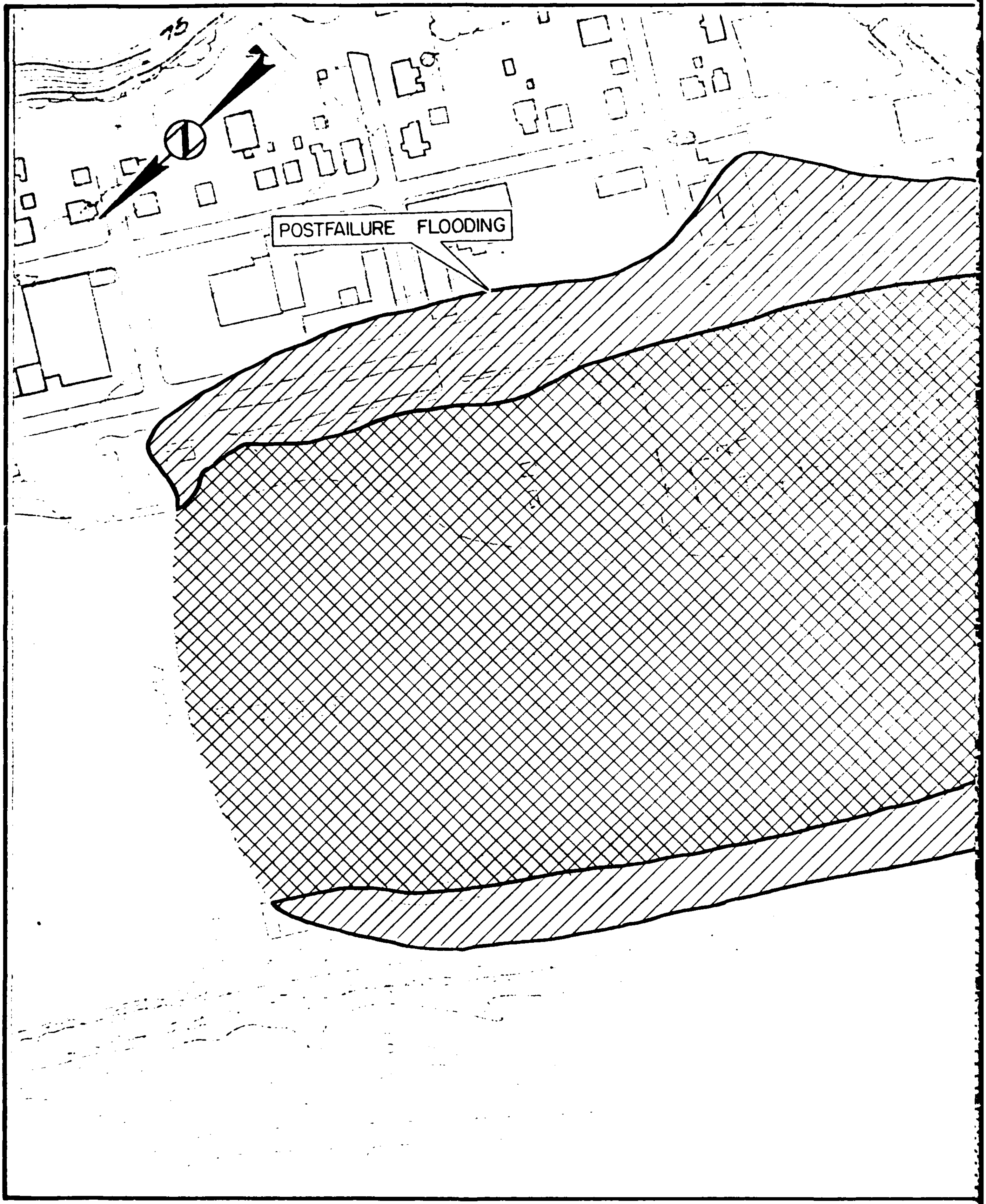
NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

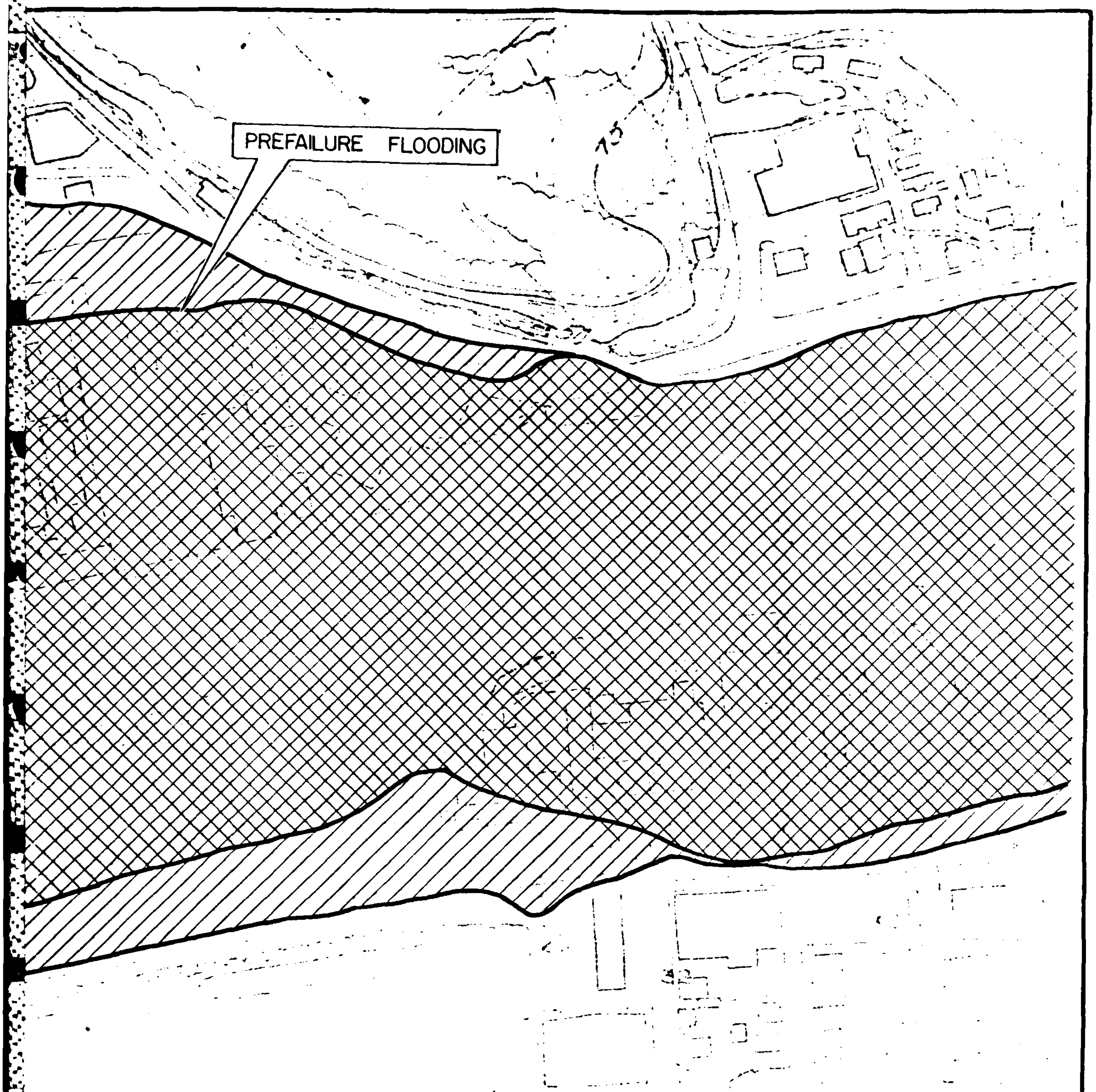
DRAINAGE AREA MAP
LAKE HOUSATONIC DAM

HOUSATONIC RIVER

SHELTON-DERBY, CONNECTICUT

DWN BY	CKD BY	APP BY	SCALE 1 250,000	
[illegible]	CHC	RAH	DATE JUL 1980	SHEET D-2


75
POSTFAILURE FLOODING



INTERNATIONAL ENGINEERING CO. DARIEN, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

TOPOGRAPHICAL MAP (1981)

HOUSATONIC RIVER — SHELTON-DERBY, CONNECTICUT

DWN BY	CKD BY	APP BY	SCALE 1" = 200'	
H. [illegible]	[illegible]		DATE JULY 1981	SHEET D-1A

INTERNATIONAL ENGINEERING COMPANY, INC.

Project NATIONAL DAM INSPECTION PROGRAM | Contract No. 2616-23 | Sheet D-1
Feature LAKE HOUSATONIC DAM | Designed | File No.
Item | Checked | Date 7-21-81
Date

HYDRAULIC / HYDROLOGIC INSPECTION

LAKE HOUSATONIC DAM, CT 00026, DERBY AND SHELTON, CONN.

DRAINAGE AREA

THE DRAINAGE AREA TRIBUTARY TO LAKE HOUSATONIC DAM IS 1574 SQUARE MILES.* HEADWATERS L[illegible] IN THE TACONIC RANGE REACHING AS FAR NORTH IN LATITUDE AS ALBANY, N.Y.. THE SOURCE RISES TO AN ELEVATION (maximum) OF SLIGHTLY OVER 3000 FEET ON BRODIE MOUNTAIN, [illegible] THE DRAINAGE BASIN IS APPROXIMATELY 91 MILES FROM SOURCE TO THE LAKE HOUSATONIC DAM, THEREBY INDICATING AN AVERAGE WIDTH OF ONLY SLIGHTLY MORE THAN 17 MILES. THE BASIN INCLUDES PART OF MASSACHUSETTS (PITTSFIELD), PORTIONS OF NEW YORK, AND MOST OF WESTERN CONNECTICUT EXCEPTING THE COASTAL DRAINAGE AREA OF SOUTHWESTERN PORTIONS OF THE STATE. THE TAILWATER OF LAKE HOUSATONIC DAM IS TIDAL. THE DAM IS APPROXIMATELY 11 MILES FROM LONG ISLAND SOUND AND 1.3 MILES UPSTREAM OF THE HOUSATONIC - NAUGATUCK RIVER CONFLUENCE. FLOW IS REGULATED BY NORTHEAST UTILITIES AT THE STEVENSON HYDROPOWER PLANT 5.5 MILES [illegible] OF LAKE HOUSATONIC DAM. THE STEVENSON DAM IMPOUNDS LAKE ZOAR.

* D.A. WAS COMPUTED FROM TWO SOURCES: "WATER RESOURCES DATA" CT, D.A. TO STEVENSON GAGE (1541 [illegible]) AND INTERMEDIATE D.A. BELOW THE GAGE, PLANIMETERED FROM USGS QUADS: ANSONIA, LONGHILL, NAUGATUCK, SOUTHBURY, AND WOODBURY (3.

INTERNATIONAL ENGINEERING COMPANY, INC.

Project
Feature Housatonic Lake Dam
Item

Contract No.
Designed
Checked

Sheet D-2
File No.
Date
Date

I Performance at Peak Flood Conditions

1) Maximum Probable Flood

a) Watershed Classified as "Rolling"

b) Watershed Area: 1574 sq. mi.

c) Peak Flood

Standard Project Flood ≅ ½ Probable Max. Rainfall

SPF = 198,000 cfs (From HUD - Flood Insurance Study, City of Shelton, March 1978)

2) Surcharge at Peak Inflow

a) Outflow Rating Curve

i) Spillway (see P. D-3)

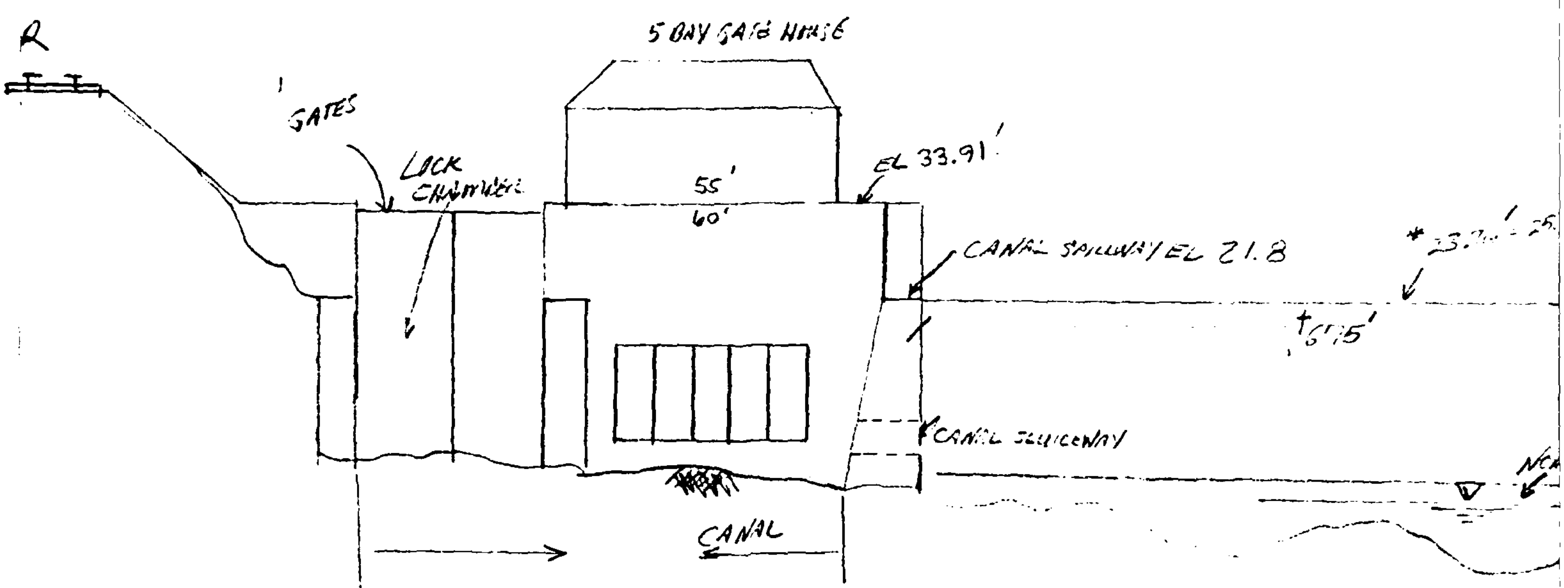
The Housatonic Lake Dam has a 675 ft free overflow spillway section. This section has been improved by significant modifications since the first time of failure during construction in October 1869. The spillway now is made of the original ashlar masonry overflow dam and a triangular masonry apron abutting

INTERNATIONAL ENGINEERING COMPANY, INC.

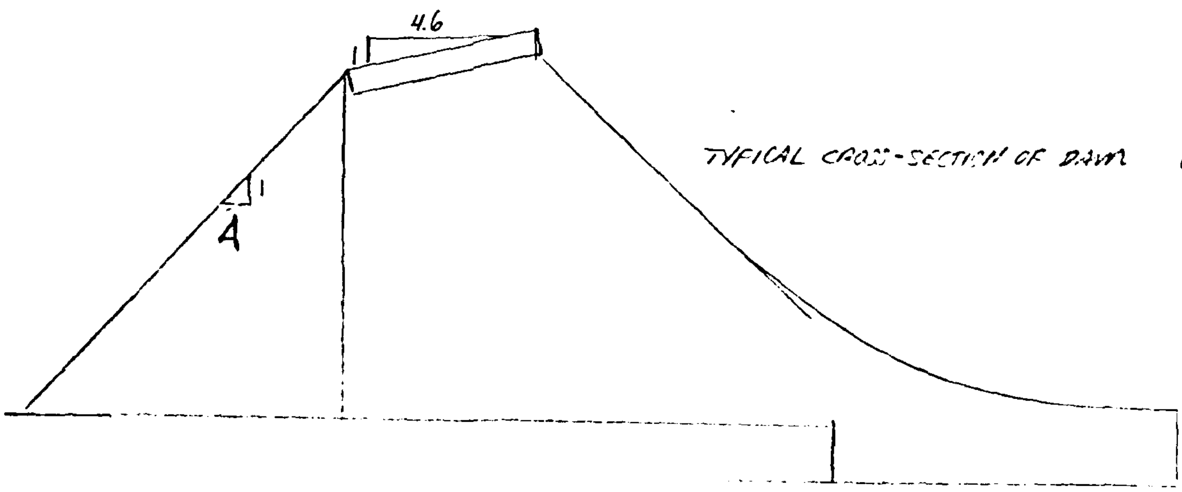
Project ______ Contract No. ______ Sheet ______ File No. ______

Feature Housatonic Lake Dam Designed ______ Date ______

Item ______ Checked ______ Date ______



Downstream Elevation



TYPICAL CROSS-SECTION OF DAM C = 3

* BASED UPON MAY 1977 SURVEY BY E.W. RICHIE (DATUM USED BY RICHIE, 1.52' ABOVE NGVD)

+ FROM REPORT ENTITLED "DERBY DAM", NU. 12-24-30 ; INCREASED FROM 537' BY 38'

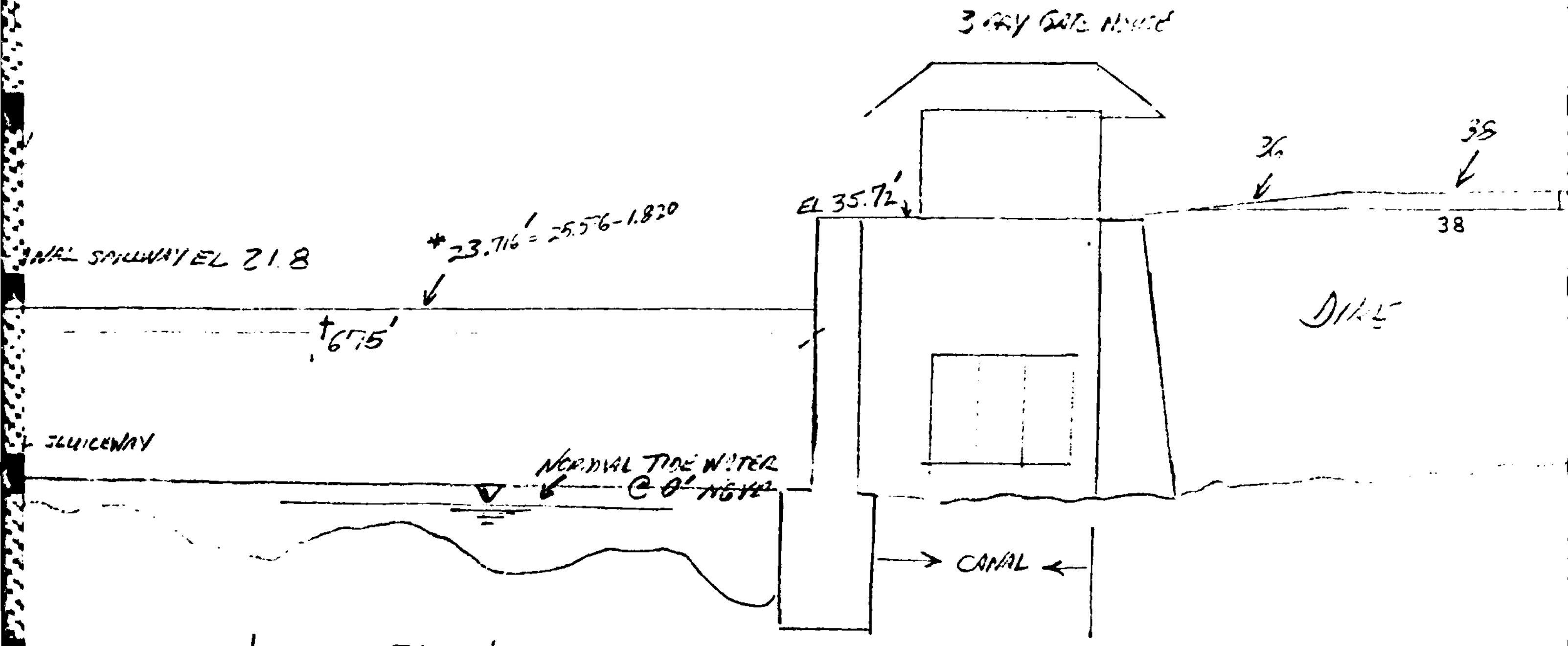
ASSUME TOP OF DAM @ EL 36' NGVD

NY, INC. Sheet ____________

No. ____________ File No. ____________

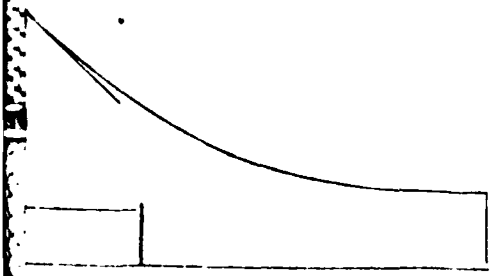
____________ Date ____________

____________ Date ____________



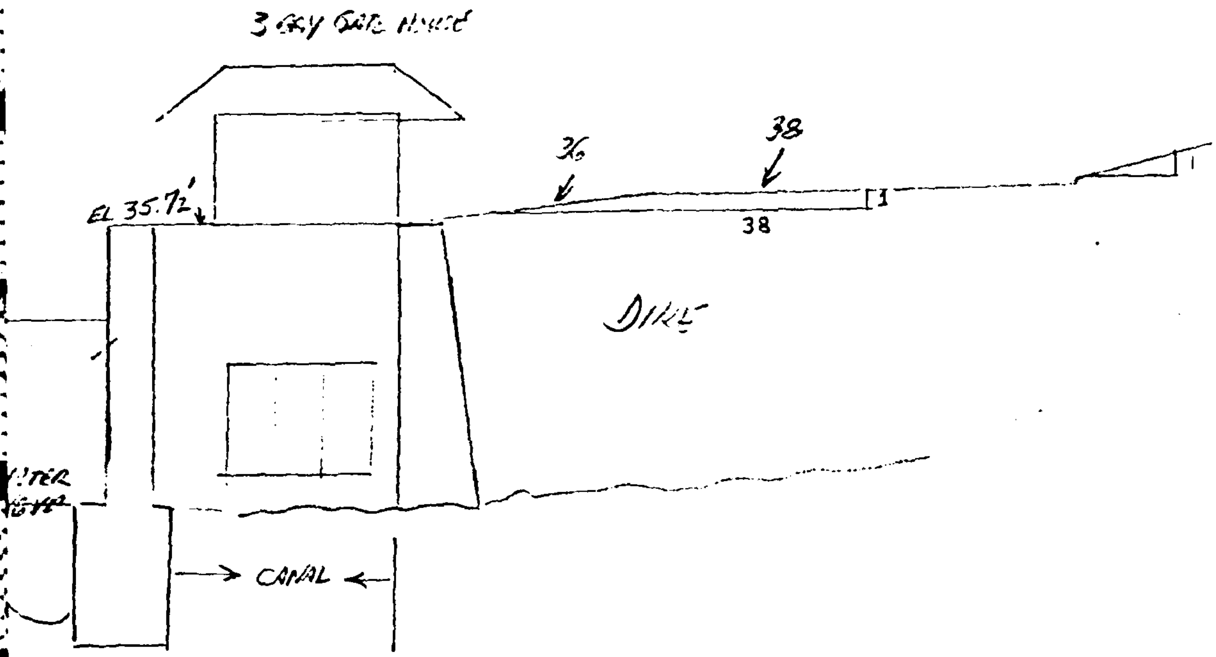
Downstream Elevation

TYPICAL CROSS-SECTION OF DAM C = 3.5 KING & BRATER 6TH ED. pps 5-26 AND 5-44



M USED BY RICKIE, 1.82' ABOVE NGVD).

30 ; INCREASED FROM 637' TO 35' FOLLOWING JAN. 22, 1891 BREAKING.



& BRATER 6TH ED. PPS 5-26 AND 5-44

JAN. 22, 1981 BREACHING.

the original section on the downstream side.
The apron's original protective timber planking has been replaced with a layer of 9 inches of concrete.
Taking the crest of the spillway as datum, the discharge equation for the spillway is:

$$Q = 3.5\,(675)\,H^{1.5} = 2363\,H^{1.5}$$

2 a)
ii) Extension of the curve for surcharges overtopping the dam and adjacent terrain.
A weir discharge coefficient of 3.5 is used for the spillway and a coefficient of 2.7 is used for weir flow over left and right adjacent terrain. The overflow section includes the earthen dike on the left upstream side of the dam. This section starts to overflow at EL. 37. An extended profile of the dam is shown below.

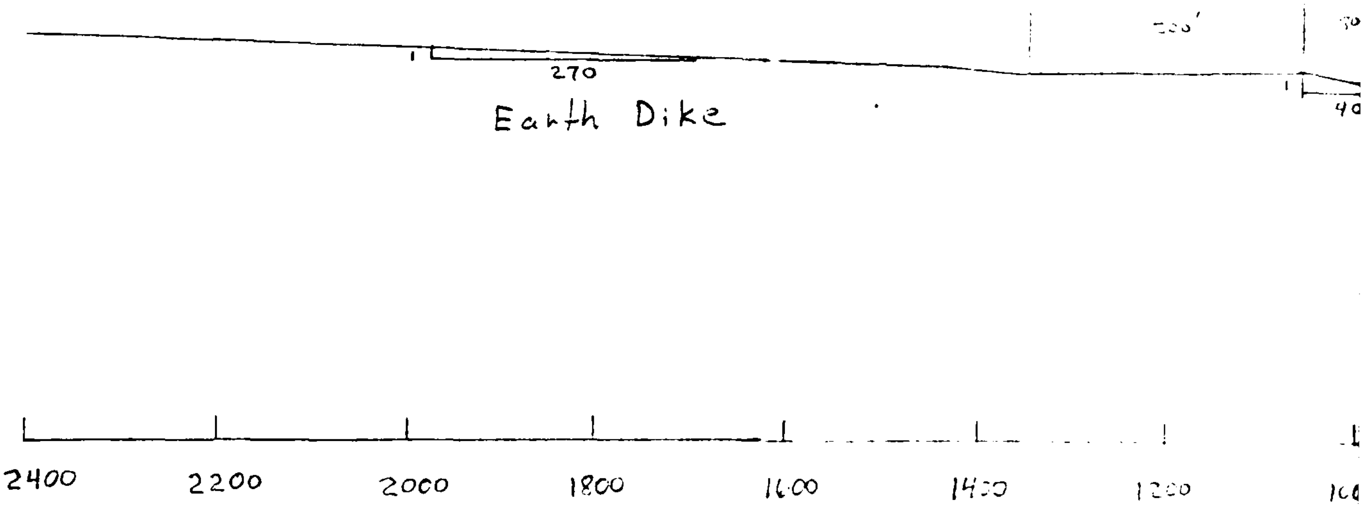
INTERNATIONAL ENGINEERING COMPANY, INC.

Project ______ Contract No. ______ Sheet ______ File No. ______

Feature Housatonic Lake Dam Designed J. Waugh Date 7/23/81

Item ______ Checked ______ Date ______

2 a)
ii)



– Discharge over Locks and Right Abutment: $q_R = (2.3)\frac{2}{5}($

$q_R = 36.8\ (H -$

– Discharge over Spillway: $q_s = 2363\ H$

– Discharge over Left Dam Abutment $q_{LDA} = 2.3\ (45$

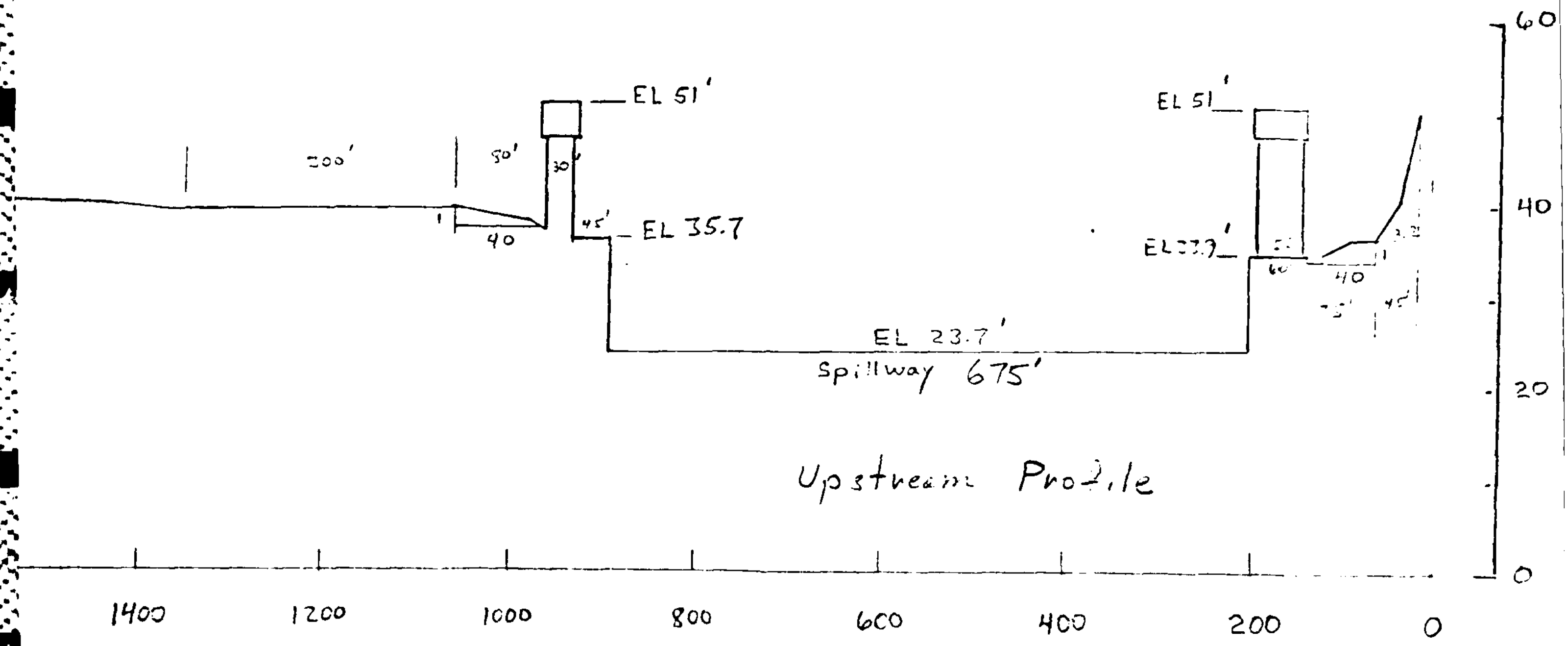
– Discharge over Earth Dike $q_{ED} = 2.3\left(\frac{2}{5}\right)$

$+\ 2.3\left(\frac{2}{5}\right)$

$q_{ED} = 36.8\ (H$

6

ANY, INC. Sheet ______
it No. ______ File No. ______
d J. Waugh Date 7/23/81
d ______ Date ______



tment: $Q_R = (2.3)\frac{2}{5}(40)(H-10.2)^{5/2}\left[1-\left(1-\frac{1/40 \times 75'}{H-10.2}\right)^{5/2}\right] + 2.3\left(\frac{2}{5}\right) \ldots 2\,(H-12$

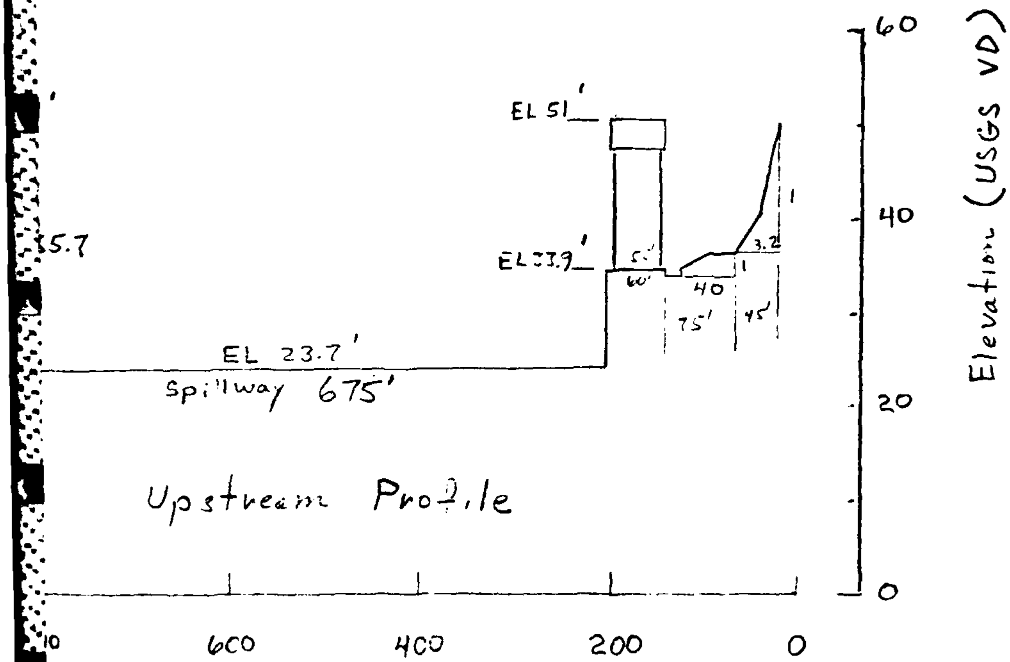
$$Q_R = 36.8\,(H-10.2)^{5/2}\left[1-\left(1-\frac{1.9}{H-10.2}\right)^{5/2}\right] + 2.9\,(H-12.1)^{5/2}$$

$$Q_S = 2363\ H^{1.5} \quad \text{(see Page D-4)}$$

$$Q_{LDA} = 2.3\,(45)(H-12.0)^{1.5} = 103.5\,(H-12.0)^{1.5}$$

$$Q_{ED} = 2.3\left(\frac{2}{5}\right)40(H-12)^{5/2}\left[1-\left(1-\frac{\frac{1}{40}\times 80}{(H-12)}\right)^{5/2}\right] + 2.3\,(300)\,(H-14)$$
$$+\ 2.3\left(\frac{2}{5}\right)270\,(H-14)^{5/2}$$

$$Q_{ED} = 36.8\,(H-12)^{5/2}\left[1-\left(1-\frac{2}{H-12}\right)^{5/2}\right] + 690\,(H-14)^{1.5} + 248.4\,(H$$



$$^{5/2}\left[1-\left(1-\frac{1/40 \times 75'}{H-10.2}\right)^{5/2}\right] + 2.3\left(\frac{2}{5}\right)3.2\,(H-12.1)^{5/2}$$

$$1-\left(1-\frac{1.9}{H-10.2}\right)^{5/2}\Big] + 2.9\,(H-12.1)^{5/2}$$

Page D-4)

$$)^{1.5} = 103.5\,(H-12.0)^{1.5}$$

$$)^{5/2}\left[1-\left(1-\frac{\frac{1}{40}\times 80}{(H-12)}\right)^{5/2}\right] + 2.3\,(300)\,(H-14)^{1.5}$$

$$4)^{5/2}$$

$$1-\left(1-\frac{2}{H-12}\right)^{5/2}\Big] + 690\,(H-14)^{1.5} + 248.4\,(H-14)^{5/2}$$

INTERNATIONAL ENGINEERING COMPANY, INC.

Sheet D-[illegible]

Project ________ Contract No. ________ File No. ________

Feature Housatonic Lake Dam Designed J. Haugh Date 7/23/81

Item ________ Checked [illegible] Date ________

[illegible] a)

ii) Total over-flow Discharge Equation

$$Q_T = 2363\, H^{1.5} + 36.8\,(H-10.2)^{5/2}\left[1-\left(1-\frac{1.5}{(H-10.2)}\right)^{[illegible]}\right]$$

$$+\ 2.9\,(H-12.1)^{5/2} + 103.5\,(H-12.[illegible])^{1.5}$$

$$+\ 36.8\,(H-12)^{5/2}\left[1-\left(1-\frac{2}{(H-12)}\right)^{5/2}\right] + 690\,(H-14)^{1.5}$$

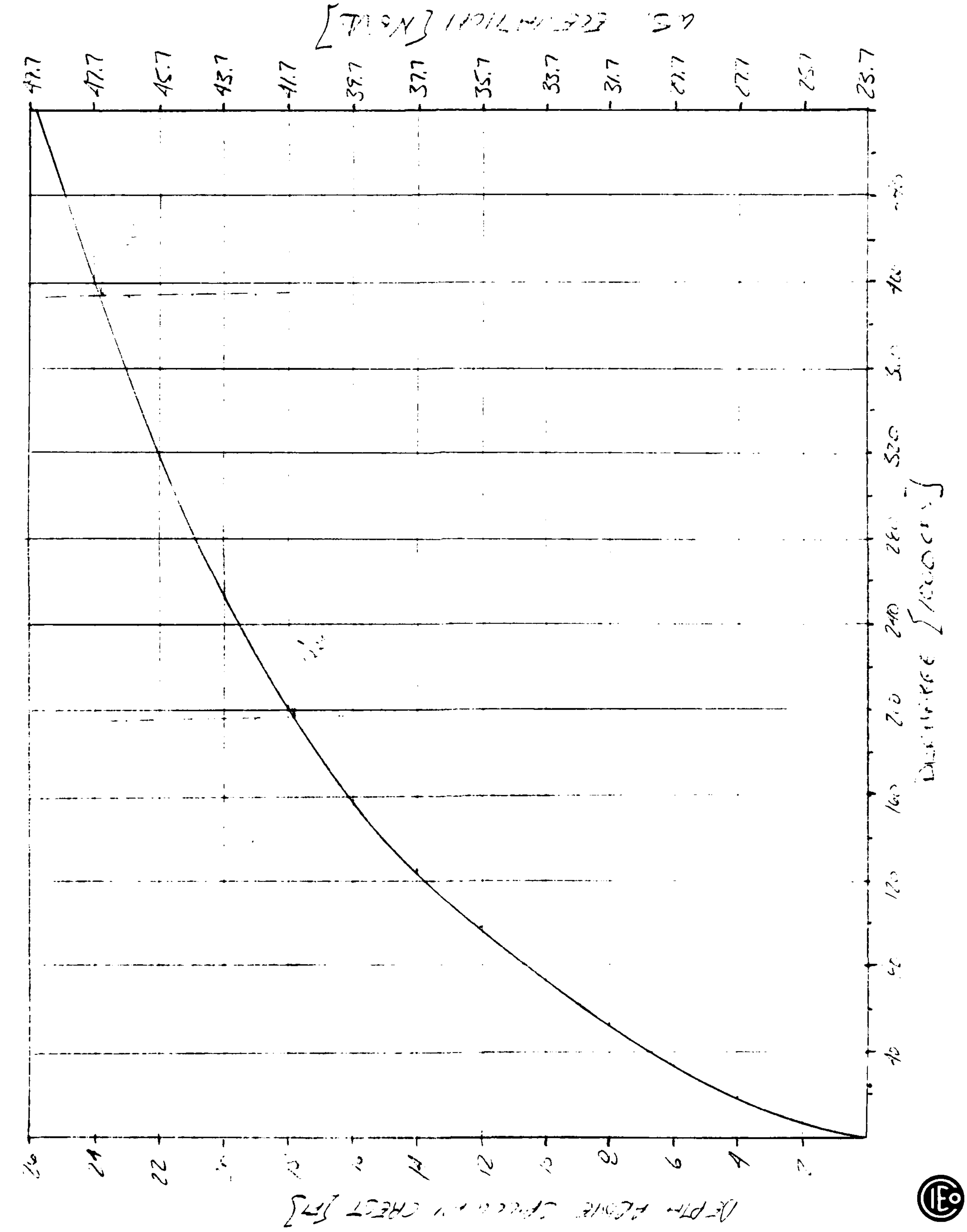
$$+\ 248.4\,(H-14)^{5/2}$$

H	Q_S	Q_R	Q_{LDA}	Q_{ED}	Q_T
2	6,680	0	0	0	6,680
4	18,900	0	0	0	18,900
8	53,500	0	0	0	53,500
12	98,200	160	0	0	98,360
14	123,800	860	300	200	125,160
16	151,200	1900	800	4,300	158,200
18	185,500	3300	1506	15,500	2[illegible],800
20	211,400	5000	2300	35,500	254,200
22	243,800	7100	3300	65,600	319,800
23	260,700	8,300	3800	84,300	357,600
25	2[illegible]5,400	10,700	4900	132,500	443,500

INTERNATIONAL ENGINEERING COMPANY, INC.

Sheet D-7

Project	Contract No. 2?16	File No.
Feature LAKE [illegible] DAM	Designed J. [illegible]	Date 7/23/81
Item	Checked [illegible]	Date 7/31/81



INTERNATIONAL ENGINEERING COMPANY, INC.

Project NDIP | Contract No. 2016 | Sheet D-9
Feature LAKE [illegible] DAM | Designed [illegible] | File No.
Item | Checked | Date [illegible]
Date

b SURCHARGE AT [illegible] TO [illegible] PEAK INFLOW

@ PMF = 396,000 CFS H = 23.9'

@ 1/2 PMF = 198,000 CFS H = 17.9'

C. EFFECT OF SURCHARGE ON PEAK OUTFLOWS

i) RESERVOIR SURCHARGE STORAGE FROM AVAILABLE TOPOGRAPHY

ii) NORMAL POOL ASSUMED AT SPILLWAY CREST

iii) DISCHARGE Q_{P_2} AT VARIOUS HYPOTHETICAL SURFACE ELEVATIONS

H = 23' V = 10,500 $S = \frac{10,500}{1574(53.3)} = .125''$

H = 21' V = 7,800 $= .093''$

H = 19' V = 6,900 $= .082''$

FROM APPROXIMATE ROUTING NED-ACE GUIDELINES AND 19 in MAXIMUM PROBABLE RUNOFF:

$$Q_{P_2} = Q_{P_1}\left(1 - \frac{S}{19}\right) \quad \text{AND} \quad Q'_{P_2} = Q'_{P_1}\left(1 - \frac{S}{9.5}\right)$$

H = 23 $Q_{P_2} = 393,400$ $Q'_{P_2} = 195,400$

H = 21 $Q_{P_2} = 394,100$ $Q'_{P_2} = 196,100$

H = 19 $Q_{P_2} = 394,300$ $Q'_{P_2} = 196,300$

INTERNATIONAL ENGINEERING COMPANY, INC.

Sheet D-7
Project NDIP Contract No. [illegible] File No.
Feature LAKE MOUNTAIN DAM Designed [illegible] Date [illegible]
Item Checked Date

d) PEAK OUTFLOWS (Q_{P_3})

USING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD AND RATING CURVE (SEE SHEET D-9).

Q_{P_3} = 394,000 CFS H_3 = 23.8'

Q_{P_3}' = 197,000 H_3' = 17.8'

3) SPILLWAY CAPACITY RATIO TO PEAK INFLOWS AND OUTFLOWS

a) SPILLWAY CAPACITY TO TOP OF DAM EL. 40.0

H = 16.3 Q_s = 155,000 CFS

∴ THE TOTAL SPILLWAY CAPACITY TO TOP OF DAM IS 39% ± OF THE PEAK INFLOW (Q_{P_1}) AND OUTFLOW (Q_{P_3}). LIKEWISE, THE TOTAL SPILLWAY CAPACITY TO TOP OF DAM IS 79% ± OF THE PEAK INFLOW (Q_{P_1}') AND OUTFLOW (Q_{P_3}').

b) SPILLWAY CAPACITY TO PMF & 1/2 PMF SURCHARGES:

i) SPILLWAY CAPACITY TO PMF SURCHARGE

H = 23.8' Q_s = 274,400 CFS

∴ THE TOTAL SPILLWAY CAPACITY TO PMF

INTERNATIONAL ENGINEERING COMPANY, INC.

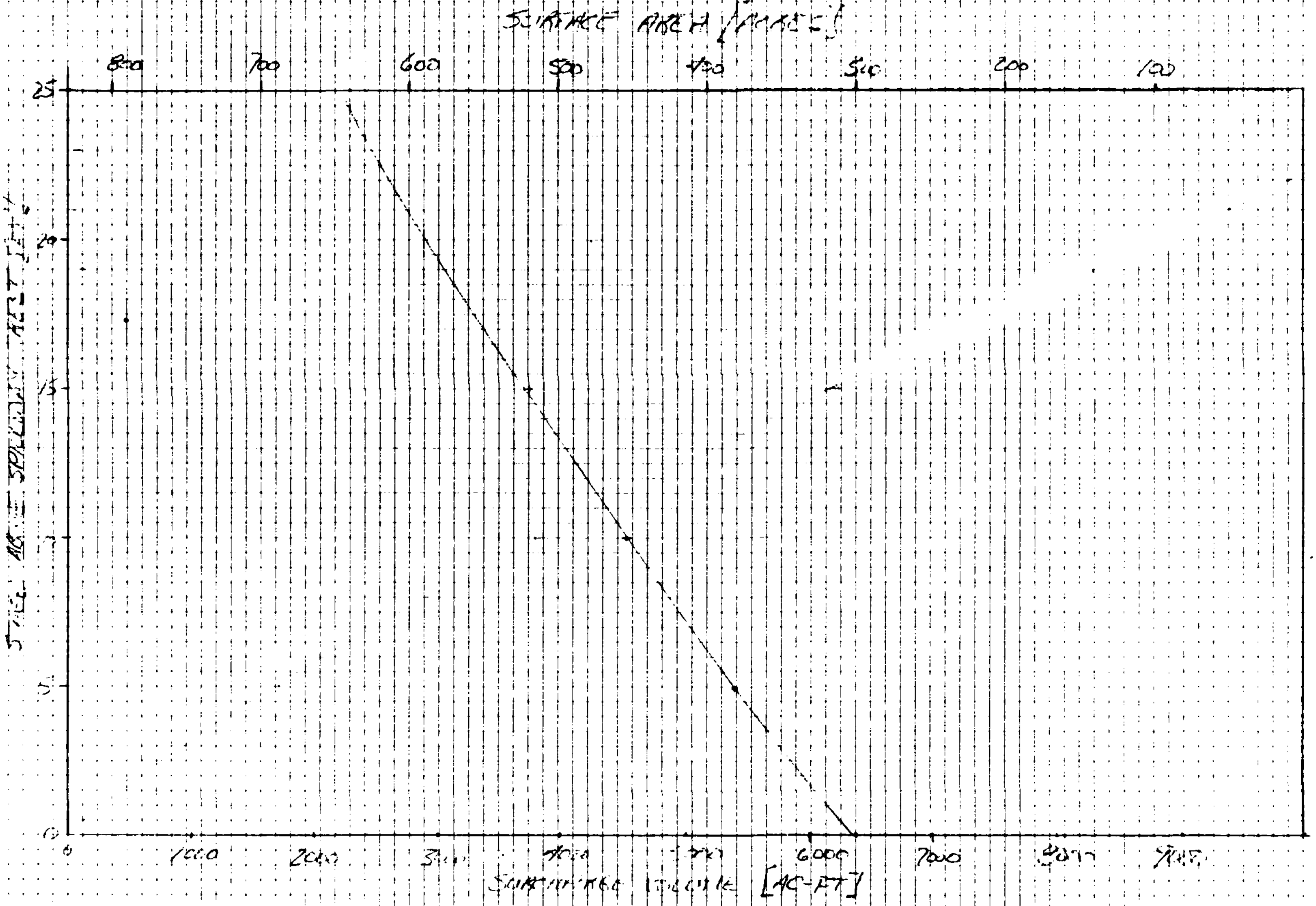
Project NDIP	Contract No. [illegible]	Sheet D-10
Feature LAKE [illegible] DAM	Designed [illegible]	File No.
Item	Checked	Date [illegible]

SURCHARGE IS 69%± OF THE INFLOW (Q_{P_1}) AND OUTFLOW (Q_{P_3}).

ii) SPILLWAY CAPACITY TO ½PMF SURCHARGE

$H = 17.8$ FT $Q = 177500$ CFS

∴ THE TOTAL SPILLWAY CAPACITY TO ½PMF SURCHARGE IS 90%± OF THE INFLOW (Q'_{P_1}) AND OUTFLOW (Q'_{P_3}).


800
700
600
500
400
300
200
100
25
20
15
10
5
0
0
1000
2000
3000
4000
5000
6000
7000
8000
9000

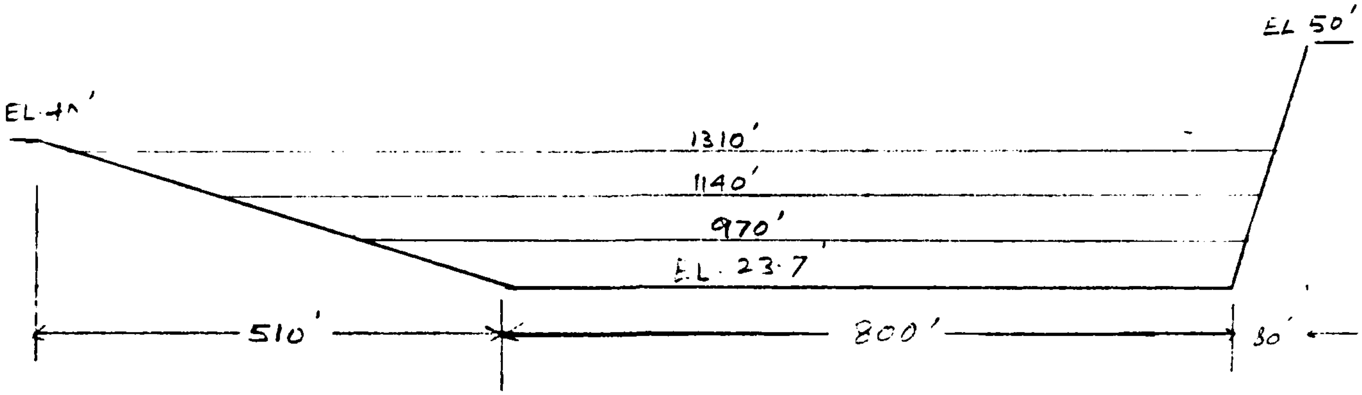
INTERNATIONAL ENGINEERING COMPANY, INC.

Sheet D-12

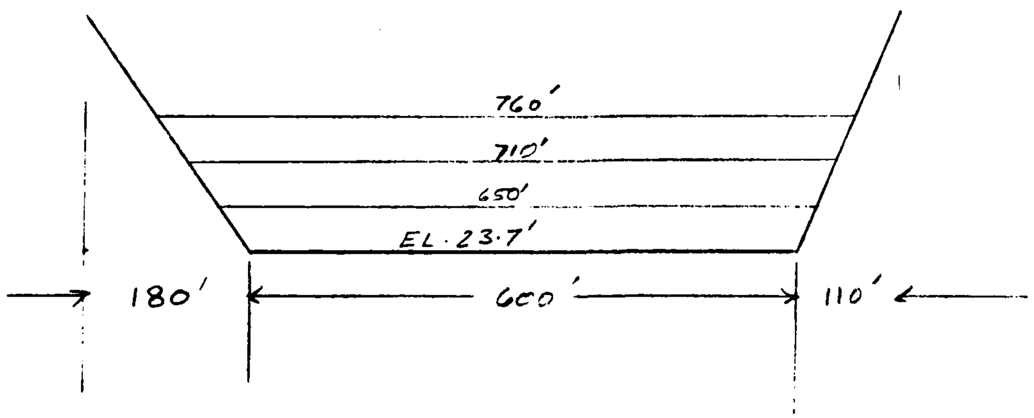
Project INSPECTION & EVALUATION OF NFD — Contract No. ______ — File No. ______

Feature LAKE HOUSATONIC DAM — Designed ______ — Date ______

Item X-SEC. OF LAKE — Checked ______ — Date ______



SEC. 1 at Dam



SEC. 2 4400' U/S FROM DAM

INTERNATIONAL ENGINEERING COMPANY, INC.

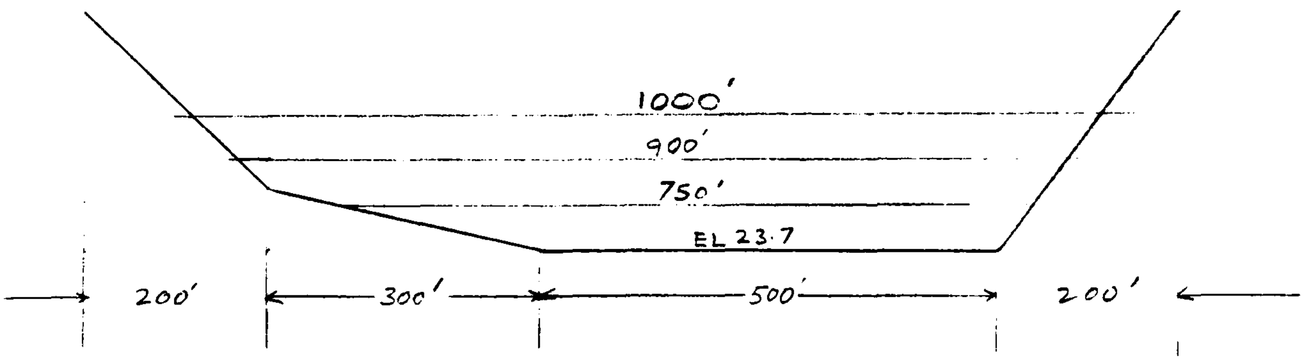
Sheet D-13

Project ______ Contract No. ______ File No. ______

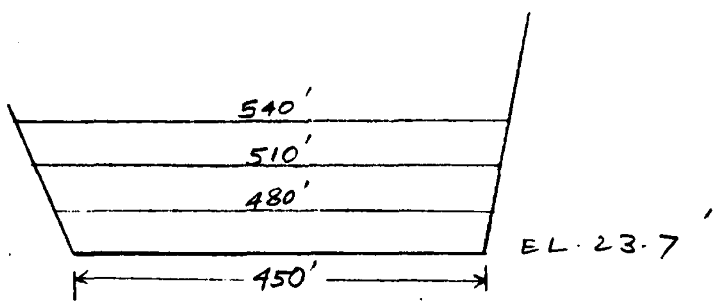
Feature ______ Designed ______ Date ______

Item ______ Checked ______ Date ______

SEC. 3 10,000 FT. U/S OF DAM



SEC. 4 16,000 FT. U/S OF DAM



INTERNATIONAL ENGINEERING COMPANY, INC.

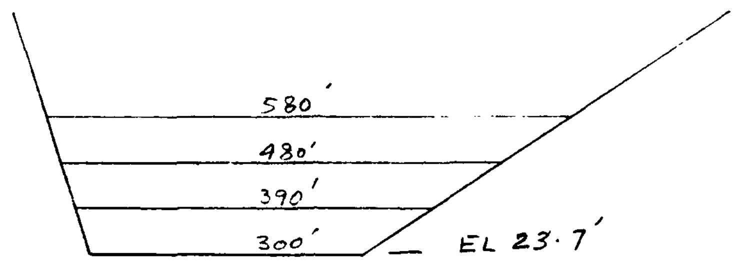
Sheet D-14

Project ______ Contract No. ______ File No. ______

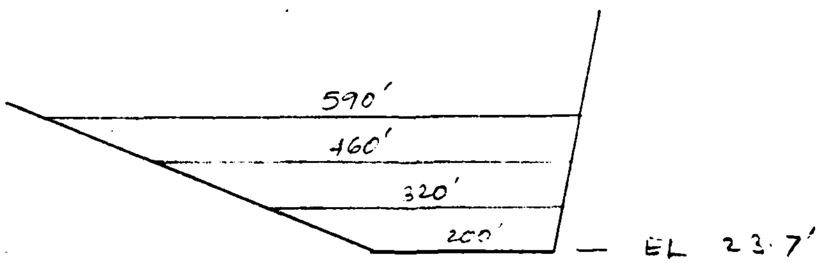
Feature ______ Designed ______ Date ______

Item ______ Checked ______ Date ______

SEC 5 21,000 FT U/S OF DAM



SEC 6 27,200 FT U/S OF DAM



SEC 7 at STEVENSON DAM — IDENTICAL WITH SEC 6

DISTANCE FROM SEC. 6 TO STEVENSON DAM = 3600'

SO DISTANCE FROM HOUSATONIC LAKE DAM TO STEVENSON DAM = 3600 + 27,200'

= 30,800'

= 5.83 miles

INTERNATIONAL ENGINEERING COMPANY, INC.

Project ______ Contract No. 2616 Sheet D-15
Feature LAKE [illegible] DAM File No. ______
Item SURCHARGE VOLUME Designed ______ Date [illegible]
Checked [illegible] Date [illegible]

STAGE 0' (CREST OF DAM, EL 23.7')

SEC. ↓	① Width (FT)	② Av. Width (FT)	③ X-SEC. AREA (FT²)	④ Av. AREA (FT²)	⑤ DISTANCE BETWEEN SECS (FT)	⑥ SURFACE AREA (Col. 2×5)	⑦ SURCHARGE VOL. (Col 4×5)
1	800'	—	—	—	—	—	—
2	600'	700'			4400'	71 Ac.	
3	500'	550'			5600'	71 "	
4	450'	475'			6000'	65 "	
5	300'	375'			5000'	43 "	
6	200'	250'			6200'	36 "	
7	200'	200'			3600'	16 "	
					TOTAL	302 Ac.	
				STAGE 5'			
1	970'	—	4425	—	—	—	—
2	650'	810'	3125	3775	4400'	82 Ac.	381 Ac. FT
3	750'	700'	3125	3125	5600'	90 "	402 "
4	480'	615'	2325	2725	6000'	85 "	375 "
5	390'	435'	1725	2025	5000'	50 "	232 "
6	320'	355'	1300	1512	6200'	50 "	215 "
7	320'	320'	1300	1300	3600'	26 "	107 "
					TOTAL	383 Ac.	1712 Ac. FT

INTERNATIONAL ENGINEERING COMPANY, INC.

Sheet D-16
Project
Contract No. [illegible]
File No.
Feature LAKE [illegible] DAM
Designed
Date [illegible]
Item SURCHARGE VOLUME
Checked [illegible]
Date [illegible]

STAGE 10'

SEC. ↓	① Width (FT)	② Av. Width (FT)	③ X-SEC. AREA (FT²)	④ Av AREA (FT²)	⑤ DISTANCE BETWEEN SECS (FT.)	⑥ SURFACE AREA (2×5)	⑦ SURCHARGE VOLUME (4×5)
1	1140'	—	9700	—	—	—	—
2	710'	925	6550	8125	4400	93 Ac.	821 Ac Ft.
3	900'	805	7000	6775	5600	103 "	871 "
4	510'	705	4800	5900	6000	97 "	813 "
5	480'	495	3900	4350	5000	57 "	499 "
6	460'	470	3300	3600	6200	67 "	512 "
7	460'	460	3300	3300	3600	38 "	273 "
					TOTAL	455 Ac.	3789 Ac FT.

STAGE 15'

SEC.	① Width (FT)	② Av. Width (FT)	③ X-SEC. AREA (FT²)	④ Av AREA (FT²)	⑤ DISTANCE BETWEEN SECS (FT.)	⑥ SURFACE AREA (2×5)	⑦ SURCHARGE VOLUME (4×5)
1	1310	—	15,825	—	—	—	—
2	760	1035	10,200	13,012	4400	105 Ac	1314 Ac Ft
3	1000	880	11,750	10,975	5600	113 "	1411 "
4	540	770	7425	9588	6000	106 "	1321 "
5	580	560	6600	7012	5000	64 "	805 "
6	590	585	5925	6263	6200	83 "	891 "
7	590	590	5925	5925	3600	49	490 "
					TOTAL	520 Ac	6232 Ac FT.

INTERNATIONAL ENGINEERING COMPANY, INC.

Project ____ Contract No. 2616 Sheet D-17

Feature Lake Housatonic Dam File No. ____

Designed ____ Date ____

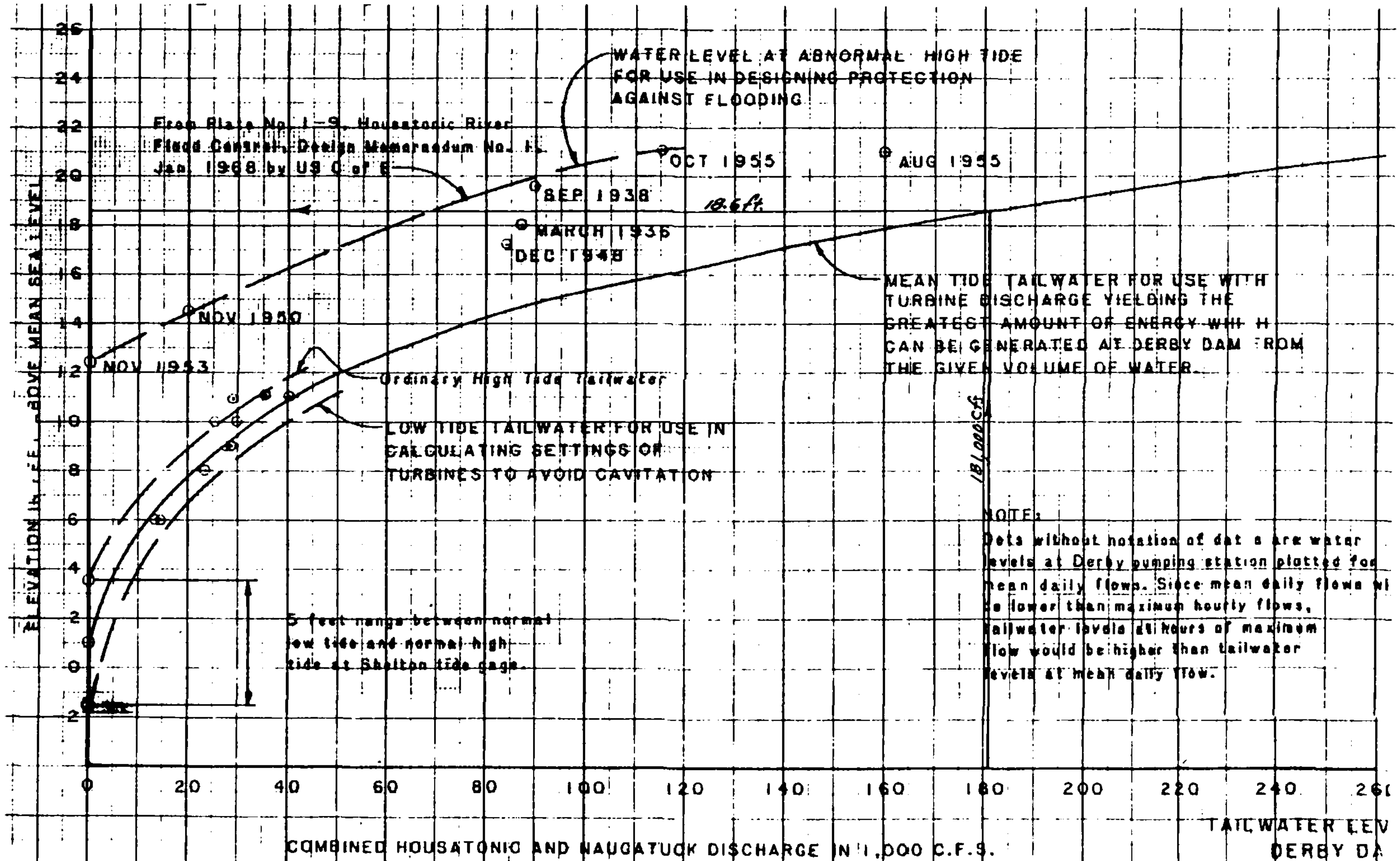
Item ____ Checked EWB Date 7/31/81

II Downstream Failure Hazard:

In order to evaluate the downstream failure hazard one must know the degree of flooding immediately prior to failure. Since the river is tidal below the Housatonic Dam the mean tide tailwater elevation was taken to be the same at the Bridge Street cross section as the tailwater elevation* at the confluence of the Naugatuck and the Housatonic Rivers produced by area wide (including the Naugatuck Basin) flood of such magnitude as to create top of dam spilling. From the Bridge Street cross section the backwater profile was computed to the dam to establish the extent of flooding before breaching of the dam (EL 40, q_{PB} = 163,000 cfs and $q_{Housat. + Naug.}$ = 181,000 cfs). The values obtained in IECO's tailwater analysis (D-19 & D-20) differed from those obtained in the Petrons tailwater curve by as much as 5 feet. The lack of adequate topography and river cross-sections from the dam to the confluence of the rivers is currently blamed for these discrepancies. See Section 7 for recommended follow-up action.

* Tidal Tailwater curve provided by Petrons, p. D-18

D-18



TAILWATER LEV
DERBY DA

ATTACHMEN

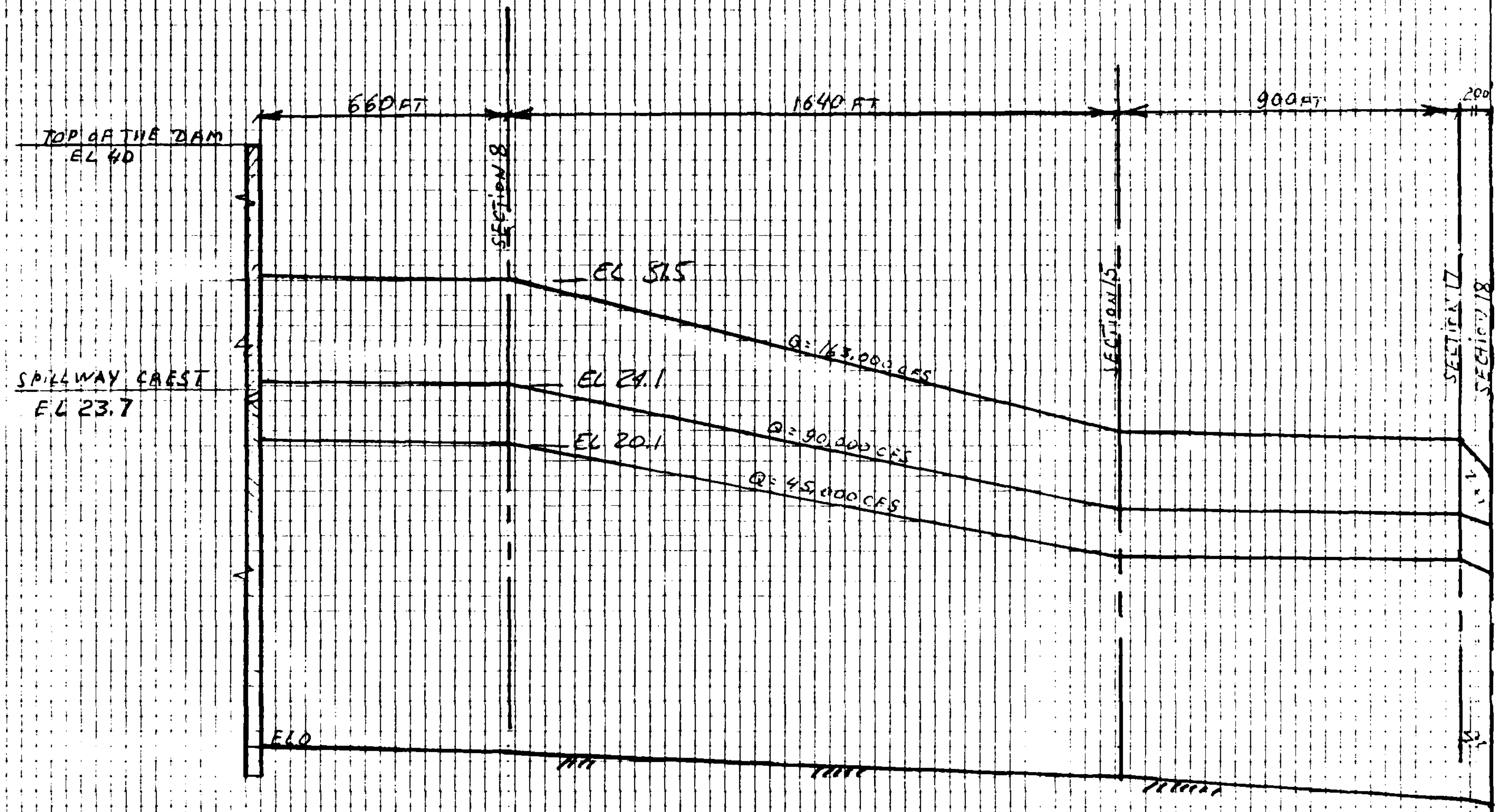
PROTRANS consultants
P.O. Box 12608

INTERNATIONAL ENGINEERING COMPANY, INC.

Project ____ Contract No. 2616 Sheet D-19
Feature HOUSATONIC LAKE DAM Designed Y.B File No. ____
Item BACKWATER CURVE Checked ____ Date 7/29/81
Date ____

SECTION	Z	Y FT	A FT^2	V FPS	$\frac{V^2}{2g}$	H FT	R FT	$R^{4/3}$ FT	S_f	$\bar{S}_f$	ΔX FT	h_f FT	H
					Q=163,000 CFS			n=0.035					
18	18	22	11,110	14.4	3.22	21.22	15.6	38.9	0.0029	–			21.22
17	20.5	24	12,015	13.6	2.85	23.35	17.7	46.1	0.0022	0.00256	800	2.04	23.24
15	21	23	8100	20.1	6.28	27.28	14.2	34.4	0.00647	0.00434	900	3.90	27.14
8	31.5	32.5	15,600	10.44	1.69	33.19	18.3	48.22	0.00124	0.00385	1640	6.31	33.44
						Q=90,000 CFS		n=0.035					
18	14.8	18.8	8,648	10.4	1.68	16.48	13.26	31.38	0.00190	–	–	–	16.48
17	16.4	19.9	9,573	9.40	1.37	17.77	14.74	36.16	0.00135	0.00162	800	1.30	17.78
15	16.9	18.9	5,800	15.5	3.74	20.64	12	27.47	0.0048	0.00311	900	2.78	20.56
8	24.1	24.6	8,000	11.25	1.96	26.08	14.7	36	0.00194	0.00337	1640	5.52	26.08
						Q=45,000 CFS		n=0.035					
18	11.4	15.4	6,468	6.95	0.75	12.15	11.93	27.25	0.00098	–	–	–	12.15
17	12.3	15.8	8,058	5.58	0.48	12.78	13.45	31.99	0.00054	0.00076	800	0.62	12.77
15	12.7	14.7	3,300	13.63	2.89	15.59	8.7	17.89	0.0057	0.00312	900	2.81	15.57
8	20.1	20.6	5,400	8.33	1.08	21.18	13.6	32.46	0.0011	0.0034	1640	5.64	21.21

THE RESULTS OF THIS STANDARD STEP ANALYSIS ARE PRESENTED GRAPHICALLY ON PG D-20. AT 163,000 cfs THE TAILWATER ELEVATION AT THE DAM IS APPROXIMATELY 31.5 NGVD. BY EXTRAPOLATING THE PROTRANS TAILWATER CURVE THE TAILWATER ELEVATION AT 163,000 cfs IS 25.8, THUS THE MORE CONSERVATIVE VALUE (25.8) WAS USED IN THIS APPENDIX.



WATER SURFACE PROFILES

INTERNATIONAL ENGINEERING COMPANY, INC.

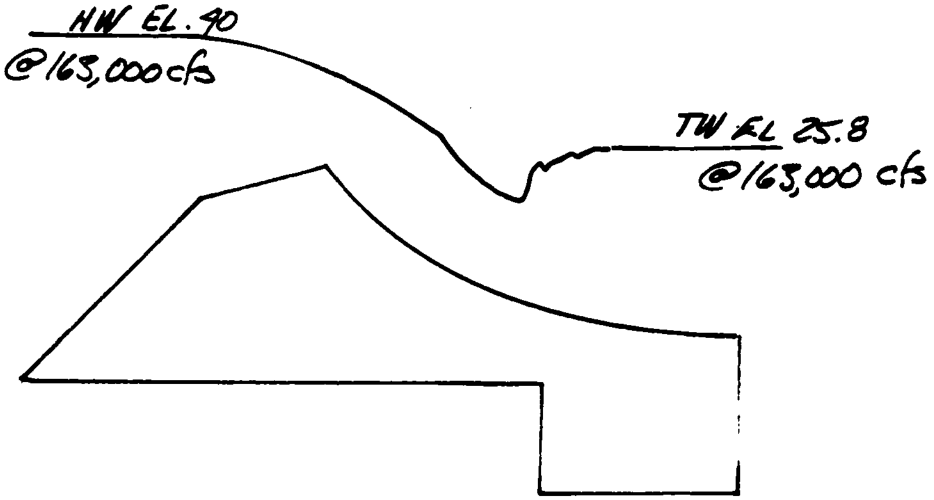
Project NATIONAL DAM INSPECTION PROGRAM
Feature LAKE HOUSATONIC DAM
Item

Contract No. 2616-023
Designed
Checked

Sheet D-21
File No.
Date 7-25-81
Date

FAILURE OF SPILLWAY

FAILURE OF THE SPILLWAY UNDER CONDITIONS OF WATER SURFACE AT THE TOP OF DAM WOULD BE CAUSED BY A MAXIMUM DIFFERENTIAL HEAD ACROSS THE DAM OF 12.6 FEET.



THIS IS LESS THAN THAT EXPERIENCED UNDER NORMAL CONDITIONS &, PROVIDING UPLIFT PRESSURES ARE NOT AFFECTED BY GREATER GROSS HEADS ON UPSTREAM AND DOWNSTREAM TOES, THE DAM WILL BE SAFER AGAINST FAILURE AT HIGH THAN AT LOWER FLOWS. HOWEVER, FAILURE OF THIS SPILLWAY IS INVESTIGATED ACCORDING TO NED - "RULE OF THUMB" DAM FAILURE PROCEDURES, PARAGRAPH d., DUPLICATED BELOW.

d. Fail dam using Y_o equal to height from invert of channel to top of dam - not from tailwater to top of dam. In a case where high tailwater highly submerges the breach (say T.W. > $0.7Y_o$) , further computation may be needed to determine a failure discharge. Obviously the tailwater resulting from the failure discharge cannot exceed the initial pool level. Therefore, under high tailwater conditions that would significantly limit the maximum failure discharge, the failure discharge might be approximated using the tailwater tating curve with allowance for an appropriate hydraulic neadloss between reservoir level and tailwater, say one velocity head: $H_1=V^2/2g$.

ASSUMING Y_o = 40 FEET, MIDHEIGHT OF DAM IS 40/2 = 20. THEREFORE, LENGTH AT MIDHEIGHT = 675'.

BREACH WIDTH = .4(675) = 270

PEAK FAILURE OUTFLOW (WITHOUT TAILWATER EFFECTS)

$$Q_B = 8/27 \sqrt{g} (270)(40)^{3/2} = 114{,}844 \text{ cfs} < 163{,}000 \text{ cfs}$$

THEREFORE, THE NET DAM BREACH FLOOD IS LESS THAN THE PRE-FAILURE DISCHARGE. THE TOTAL FLOOD IS 114,800 + 0.6(163,000) = 212,600 cfs. THIS WILL BE CHECKED USING A MOMENTUM APPROACH. USING THE CRITERIA QUOTED ABOVE, SPECIFICALLY THE LAST SENTENCE, FROM THE TAILWATER RATING CURVE, AT A DISCHARGE OF 163,000 cfs THE TAILWATER WOULD BE AT APPROXIMATELY ELEVATION 25.8*. THE VELOCITY HEAD WOULD BE, BASED UPON HYDRAULIC PARAMETERS AT SECTION-7, (SEE PAGE D-28) 1.19. ASSUMING NO FLOW

* FROM HUD FLOOD INSURANCE STUDY, BUT ADJUSTED FOR NORMAL TAILWATER CONDITIONS.

INTERNATIONAL ENGINEERING COMPANY, INC.

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Feature: LAKE HOUSATONIC DAM. — Designed: S — Date: 7-25-81

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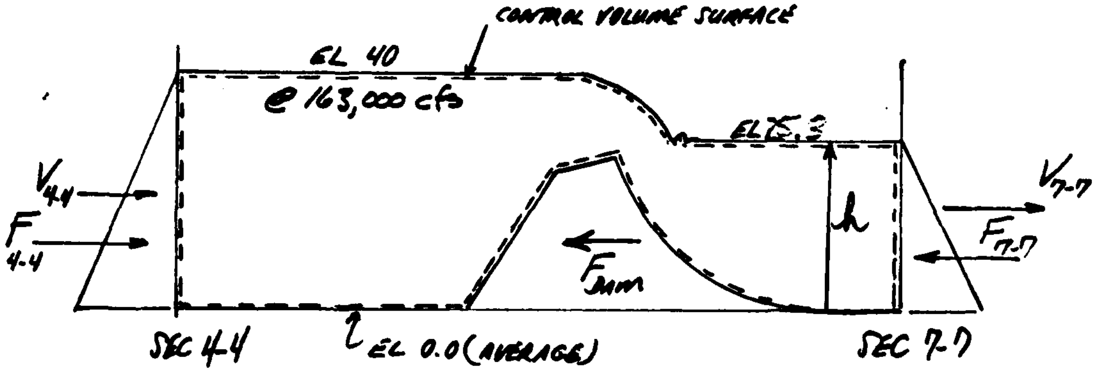
ALONG THE OVERBANKS, THE FLOW AREA IS 18,600 @ EL25.3 IN SECTION-7-7. THE AVERAGE VELOCITY, 8.8. THE AVERAGE VELOCITY UPSTREAM IN SECTION-4-4 IS 4.08. USING THE MOMENTUM PRINCIPLE,



BEFORE FAILURE, PER UNIT LENGTH OF DAM,

$$F_{44} - F_{77} = \rho Q(V_{7\text{-}7} - V_{4\text{-}4}) = \rho Q \Delta V$$

ASSUMING NO NET HORIZONTAL COMPONENT DUE TO GRAVITY. AT THE TIME OF BREACH, AN APPROXIMATE ASSUMPTION USED HEREIN, IS THAT THE FLOW ACROSS SEC 4-4 REMAINS CONSTANT WHILE FLOW ACCELERATES D/S OF THE DAM. BECAUSE THIS IS AN UNSTEADY CONDITION, OUTFLOW FROM THE CONTROL VOLUME > INFLOW OVER THE PERIOD OF TIME REQUIRED TO GENERATE THE BREACH PEAK OUTFLOW.

PER-UNIT LENGTH, $F_{4\text{-}4} = \dfrac{\gamma (40)^2}{2}$

$$F_{77} = \frac{\gamma h^2}{2}$$

INTERNATIONAL ENGINEERING COMPANY, INC.

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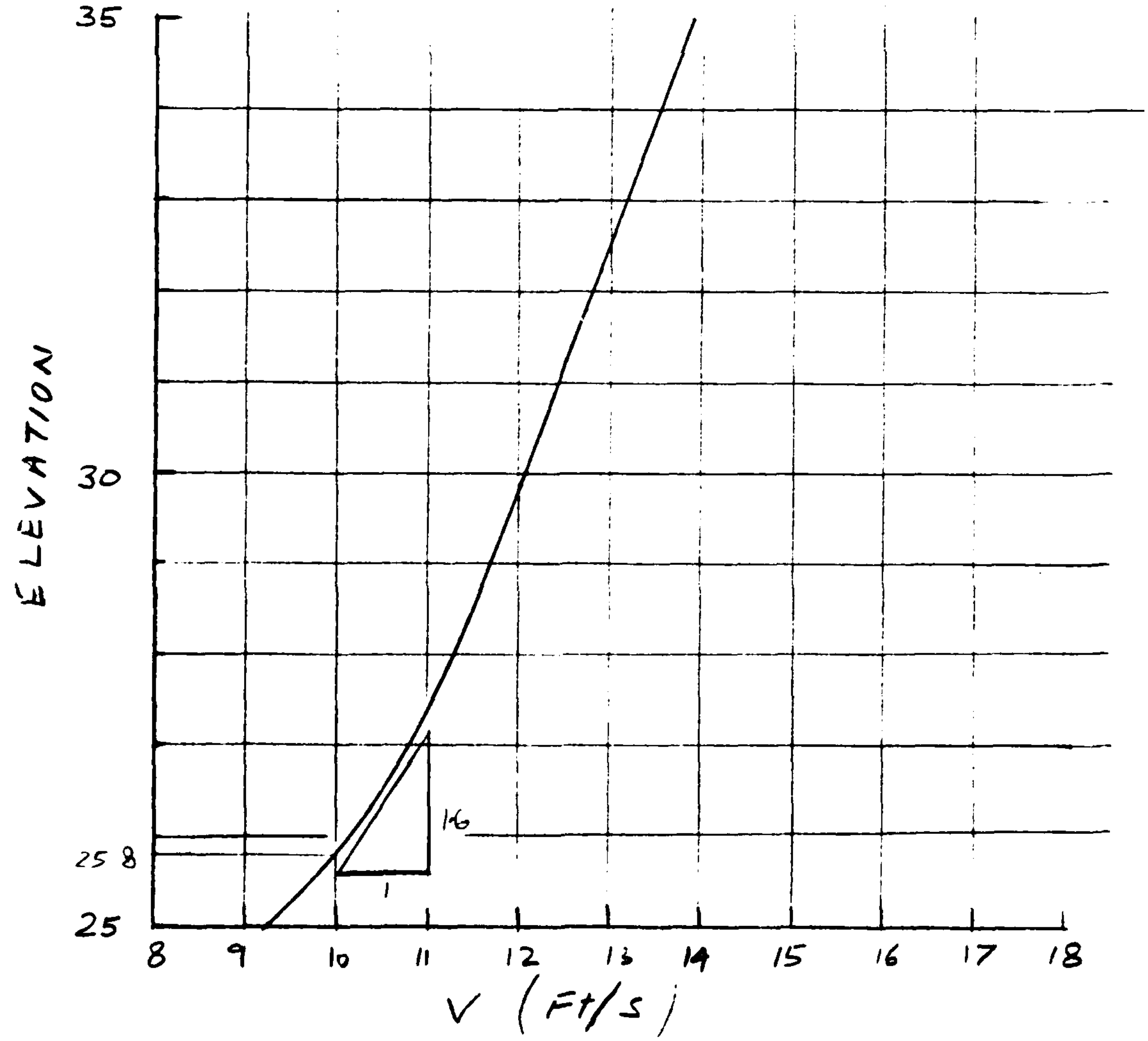
$V_{4\text{-}4} = 4.08$ $\qquad$ $Q_{4\text{-}4} = 163{,}000$

V_{77} IS A FUNCTION OF h $\qquad$ $Q_{7\text{-}7}$ IS A FUNCTION OF h

THEREFORE

$$F_{4\text{-}4} - F_{7\text{-}7} = \rho Q_{7\text{-}7} V_{7\text{-}7} - \rho Q_{4\text{-}4} \cdot V_{4\text{-}4}$$

FROM THE TAILWATER CURVE P D-26, AND STAGE-AREA RELATIONSHIP, P. D-27, A PLOT OF VELOCITY VERSUS STAGE CAN BE DEVELOPED, AND IS GIVEN BELOW ASSUMING A LINEAR VARIATION IN DISCHARGE WITH STAGE, EXTRAPOLATING FROM THE UPPER



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LIMB OF THE TAILWATER RATING CURVE AT HIGH STAGES,

DISCHARGE IS ESTIMATED TO INCREASE 20,000 cfs per foot.

ELEVATION	AREA	VELOCITY	Q
25	16,200	9.25	150,000
30	20,400	12.25	250,000
35	25,200	13.89	350,000

FROM THE GRAPH, V INCREASES BY 0.6 FT PER SECOND PER ONE FOOT INCREASE IN STAGE PER UNIT LENGTH OF DAM.

$$\frac{\gamma (40)^2}{2} - \frac{\gamma h^2}{2} = 1.94 \left[V_{7-7}^2 \cdot h - (4.08)^2 40 \right]$$

LETTING $V_{7-7} = 10 + (h - 25.8)\,0.6$ AND SOLVING FOR h by TRIAL AND ERROR

$h = 34.15$, $V = 15$ FT/SEC

BREACH WIDTH = 270 FT

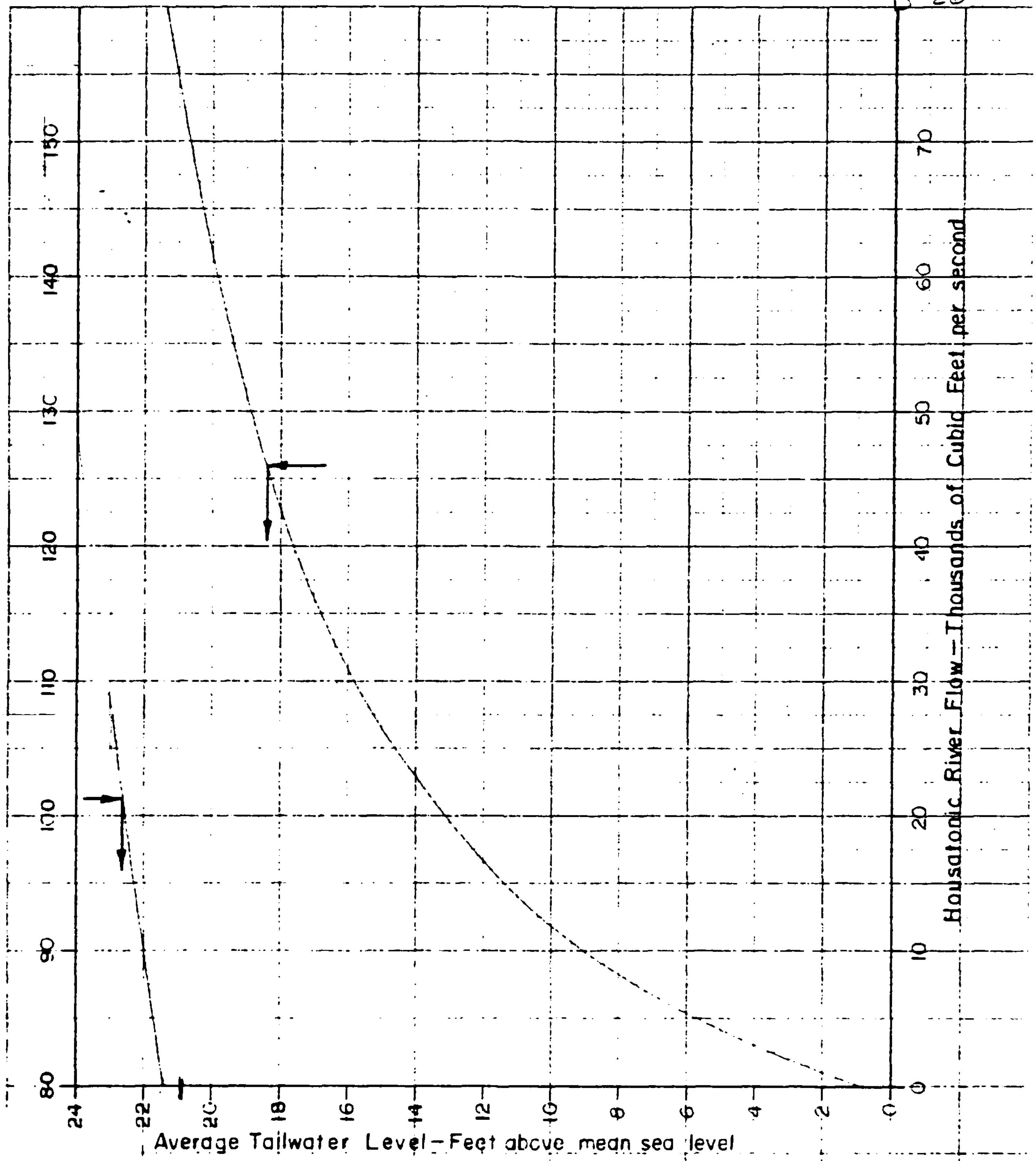
NET INCREASE IN DISCHARGE

$(15-10)(270)(34.15) = 46,000$ cfs

THEREFORE, $Q_B = 163,000 + 46,000 = 209,000$ cfs

AT A STAGE OF 27.95 @ 209,000 cfs or MAXIMUM INCREASE OF $27.95 - 25.8 = 2.15$ FT

D-26



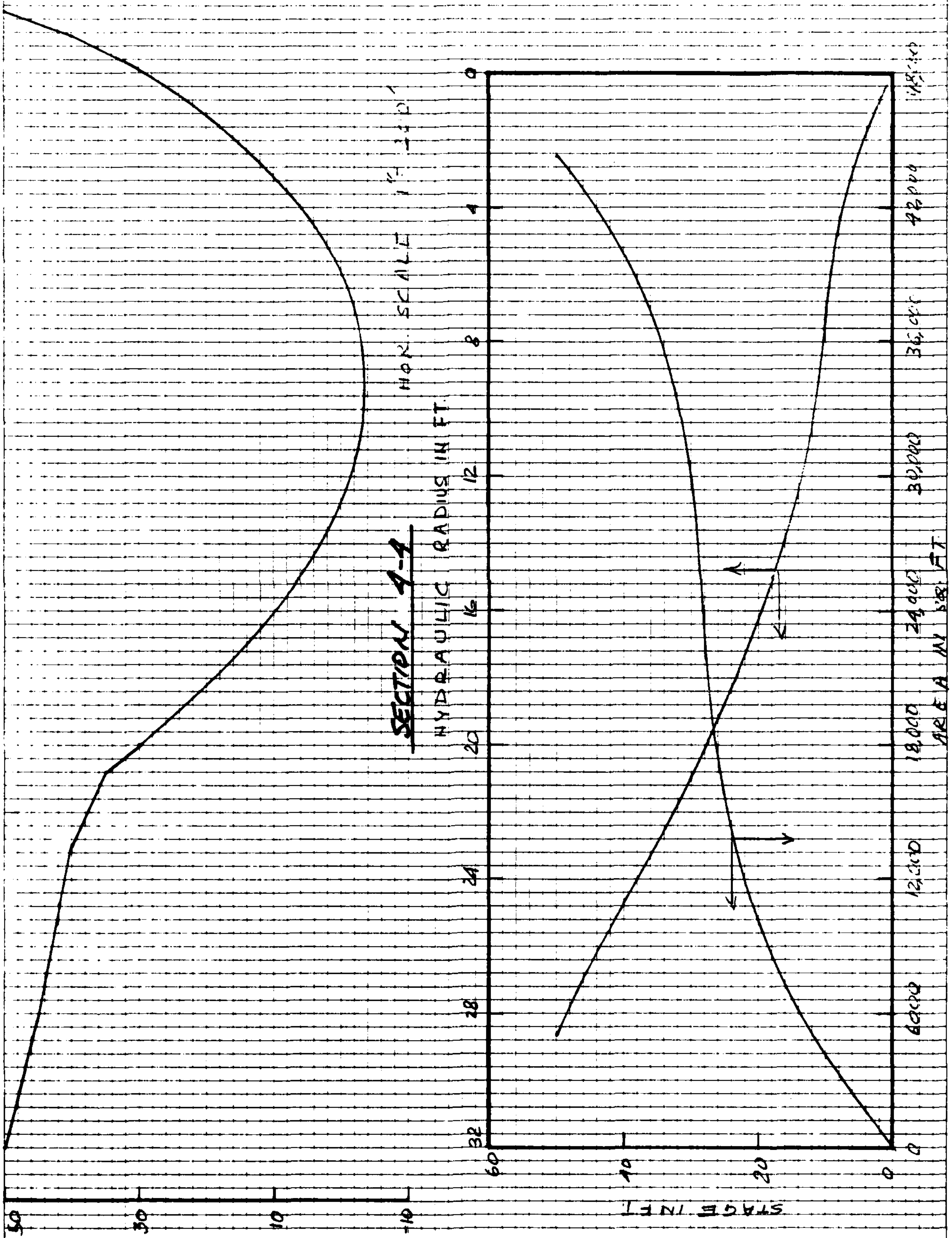
NORTHEAST UTILITIES SERVICE COMPANY
POWER PLANT AT DERBY DAM
TAILWATER RATING

PROTRANS consultants
P.C. Box 12608
El Paso, Texas 79912

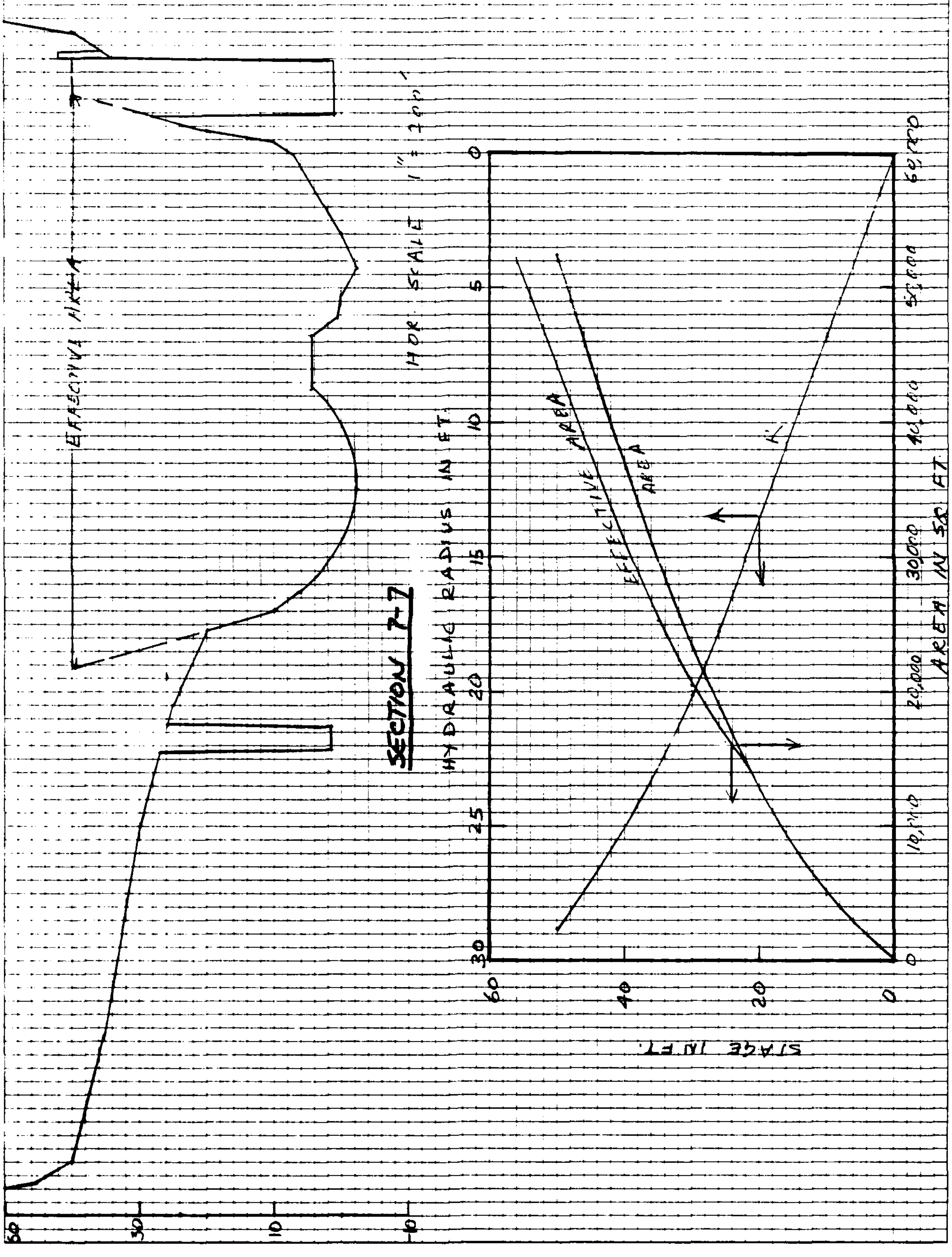
DEC. 28, 1979

Exhibit 1

• RIVER CROSS-SECTION LOCATED 1,400 FT U/S OF DAM CREST.



• RIVER CROSS-SECTION LOCATED 150 FT D/S OF DAM CREST.



POSSIBLE FAILURE OF DIKE: (SEE p. D-31 FOR PLAN LOCATION)

ASSUMING TOP OF DAM AT ELEVATION 40, FLOW WILL OCCUR OVER A PORTION OF THE DIKE UPSTREAM OF THE LEFT ABUTMENT AND GATE HOUSE. IF FAILURE OCCURS, A SURGE WILL CROSS ROOSEVELT DRIVE APPROXIMATELY 4 TO 5 FEET IN HEIGHT ABOVE STREET LEVEL TRAVELLING AT ROUGHLY 10-15 FEET/SEC; IMPACT WITH THE BUILDINGS 150' TO 200' FROM THE DIKE WILL BE SEVERE CAUSING CONSIDERABLE DAMAGE. WATER MAY EXCEED THE LEVEL OF THE ~~DIKE~~ INITIAL SURGE TEMPORARILY BECAUSE OF REFLECTION OF THE WAVE, ~~AND~~ RUNUP OF THE WAVE UPGRADE, AND SUDDEN CHANGES IN MOMENTUM ARISING FROM IMPACTS. QUANTIFICATION OF THIS PHENOMENON IS BEYOND THE SCOPE OF A PHASE I STUDY.

ASSUMING EVACUATION OF AREAS AFFECTED BY ~~RA~~ PRE-FAILURE DISCHARGE FLOODING DOWNSTREAM, A SUDDEN FAILURE OF THE DIKE IS DEEMED TO BE MORE SERIOUS. THE HAZARD IS HIGH SINCE MAXIMUM PRE-FAILURE STAGE ELEVATIONS ARE BELOW THE POTENTIAL IMPACT AREA (27.4) AS OPPOSED TO POSSIBLE, TRANSIENT STAGE ELEVATION DUE TO

INTERNATIONAL ENGINEERING COMPANY, INC.

Project NATIONAL DAM INSPECTION PROGRAM Contract No. 2616-023 Sheet D-30
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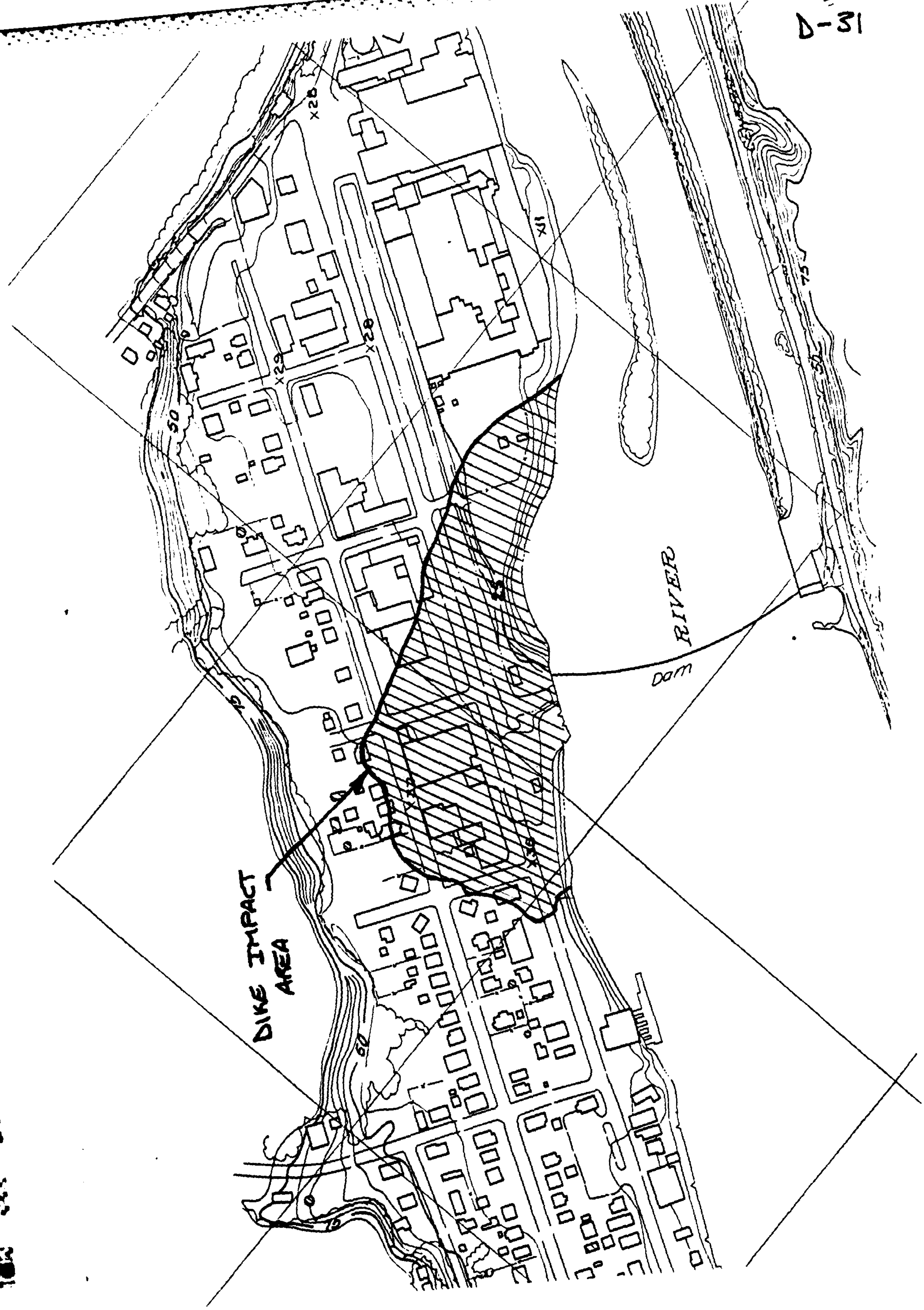
Item Checked SB Date 7-20-81 Date 7/31/81

A SUDDEN DIKE BREACH (40$^+$ FEET WITH RUNUP AND SURGE IMPACT AND REFLECTION), ON MORE THAN TWO BUILDINGS


DIKE IMPACT AREA
RIVER
Dam

INTERNATIONAL ENGINEERING COMPANY, INC.

Sheet 32

Project NJ/P — Contract No. 2616 — File No.

Feature LAKE MUSCATATIC(?) DAM — Designed EHB(?) — Date 3/1?

Item — Checked — Date

III SELECTION OF TEST FLOOD

1) CLASSIFICATION ACCORDING TO NED- ACE

a) SIZE: STORAGE (TOP OF DAM) = 10,900 AC-FT

HEIGHT ≈ 40 FT

∴ SIZE INTERMEDIATE

b) HAZARD POTENTIAL: BASED ON D/S FAILURE ANALYSIS AT LEAST 6 COMMERCIAL BUILDINGS WILL EXPERIENCE PRE FAILURE FLOODING. THE BREACH OF THE SPILLWAY WILL INUNDATE 3 ADDITIONAL COMMERCIAL BUILDINGS AND THE FAILURE OF THE DIKE WOULD FLOOD AT LEAST 9 BUILDINGS; NO PREFAILURE DOWNSTREAM OF THE DIKE — ANTICIPATED.

∴ HIGH HAZARD CLASSIFICATION

2) TEST FLOOD : PMF

APPENDIX E

INFORMATION AS CONTAINED IN THE

NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME

END

FILMED

8 84

DTIC